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Screening/Flotation Treatment of Combined Sewer Overflows. Volume II Full-Scale Operation, Racine, Wisconsin

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16. ABSTRACT

This study involved the planning, design, construction and operation of a two-year evaluation period, of three full-scale demonstration systems for the treatment of storm generated discharges. As part of the evaluation, the quality of the receiving body was also monitored. Two of the systems, located at two major points of combined sewer overflow to the Root River in Racine, Wisconsin, employed screening/dissolved-air flotation to treat the overflow prior to discharge. The two systems had a combined capacity of 222,000 cu m/day (58.5 mgd). The third system utilized screening only for the treatment of urban stormwater, having a capacity of 14,800 cu m/day (3.9 mgd).

Results indicated that the "satellite plant" concept of locating treatment plants at points of combined sewer overflow discharge is a feasible alternative to combined sewer separation. Based on the operating results of these systems, removal efficiencies of 60 to 75 percent can be expected for suspended solids and 50 to 65 percent for BOD. The chlorination system met the fecal coliform standard for the whole body contact specified by the State of Wisconsin for the Root River. It was concluded that the operation of the treatment systems had a beneficial effect on the quality of the River.

Results from the screening of urban stormwater indicated that this method would remove 50 percent of the suspended solids and 20 percent of the BOD.

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August 1979

SCREENING/FLOTATION TREATMENT OF COMBINED SEWER OVERFLOWS

VOLUME II: FULL-SCALE OPERATION RACINE, WISCONSIN

by

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FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problems, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies, and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

This report describes the planning, design and construction and the operation over a two-year evaluation period of three full-scale demonstration systems for the treatment of storm generated discharges using the screening/flotation principle.

Francis T. Mayo
Director
Municipal Environmental
Research Laboratory

ABSTRACT

This report describes the planning, design and construction and the operation over a two-year evaluation period, of three full-scale demonstration systems for the treatment of storm generated discharges. As part of the evaluation, the quality of the receiving body was also monitored. Two of the systems - located at two major points of combined sewer overflow to the Root River in Racine, Wisconsin - are identical in concept and employ screening/dissolved-air flotation to treat the overflow prior to discharge. The two systems have a combined capacity of 222,000 cu m/day (58.5 mgd). The third system utilizes screening only for the treatment of urban stormwater. It has a capacity of 14,800 cu m/day (3.9 mgd).

This report also describes the verification and modification of the EPA Storm Water Management Model using the subject drainage area, sewerage and treatment systems, and receiving body.

Results from the evaluation program indicate that the "satellite plant" concept of locating treatment plants at points of combined sewer overflow discharge is a feasible alternative to combined sewer separation. Based on the operating results for these systems, removal efficiencies of 60 to 75 percent can be expected for suspended solids and 50 to 65 percent for BOD. The chlorination system met the fecal coliform standard for whole body contact specified by the State of Wisconsin for the Root River. It was concluded that the operation of the treatment systems had a beneficial effect on the quality of the River.

Results from the screening of urban stormwater indicate that this method will remove 50 percent of the suspended solids and 20 percent of the BOD.

This report was submitted in fulfillment of Grant No. S800744 (formerly 11023 FWS) under the partial sponsorship of the Environmental Protection Agency. The study program associated with this project was performed by Envirex Inc. acting as a subcontractor to the grantee, the City of Racine. Work was completed as of November 1974.

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Field operation of the treatment units during more than fifty storm events, quickly responded to at all hours of the day and night, was performed by such dedicated individuals as John Moser, Michael Clark, Richard Race, David Gruber and Thomas Meinholz.

Timely completion of the laboratory analyses for the storm events over the two year monitoring period was achieved consistently by Richard E. Wullschleger and his laboratory staff.

The cooperation of the City of Racine in the day to day operation and maintenance of the treatment facilities is duly recognized.

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SECTION I

CONCLUSIONS

SECTION IV, TREATMENT SITES

1. Based on storm-generated discharge measurements and on quality determinations of the discharge and of the river during 1971, the discharge of untreated combined sewage and storm water was a major source of pollution of the Root River within the City of Racine. For example, the concentration of fecal coliform bacteria near the mouth of the river during a 6-hr period after a storm, averaged more than 40 times that during dry weather.

2. Based on a water quality survey and preliminary river modeling, the maximum benefit, in terms of the navigable portion of the river, would result from treatment of discharges in the lower reach of the river. Significant points of overflow and potential sites within the reach were investigated. The most cost-effective potential site was in the area of Main and Dodge Streets near downtown Racine and the three demonstration systems were constructed there. Two systems, referred to as Site I and II in this report, employ screening/dissolved-air flotation for treatment of combined sewer overflow and have design treatment capacities of 53,500 cu m/day (14.3 mgd) and 168,000 cu m/day (44.4 mgd), respectively. The third system is adjacent to Site II and is referred to as Site IIA. At this Site, screening only is used for the treatment of a storm sewer discharge.

3. The mean quality characteristics found in 1971 were:

CSO QUALITY CHARACTERISTICS

		Combined sewer overflow				Storm sewer discharge	
		Site I		Site II		Site IIA	
Item	Parameter	1971	1974	1971	1974	1971	1974
BOD	mg/l	79	93	212	110	39	15
TOC	mg/l	98	95	238	122	51	46
SS	mg/l	298	266	669	661	445	376
Fec. Coli	No./100 ml	10,300	609,000	10,100	416,000	23	580

4. Using the data collected over the two-year evaluation period, the combined sewer overflow volumes were related to rainfall by the following equations:

$$V_1 = (15{,}502 \times R) - 3{,}270$$

$$V_2 = (31{,}770 \times R) - 8{,}879$$

where, V_1 = overflow volume at Site I, cu m
V_2 = overflow volume at Site II, cu m
R = total rainfall, cm.

5. Overflow volumes bypassing the plants during operation were related to the rainfall by the following equations:

$$BV_1 = 671R - 517$$

$$BV_2 = 3454 + 12{,}737E - 5873$$

where, BV_1 = plant bypass volume at Site I, cu m
BV_2 = plant bypass volume at Site II, cu m
R = total rainfall, cm
E = average rainfall intensity, cm/hr

6. Pollutant removals (concentration basis) by the screening systems alone were:

	Percent removal		
	Site I	Site II	Site IIA
BOD	28	42	20
TOC	38	44	41
Suspended Solids	32	36	50

7. The screening/dissolved-air flotation process is a feasible method for abating storm-generated discharges by treatment in full-scale applications. This conclusion is substantiated by the overall percent removals (concentration basis) achieved by the screening/dissolved-air flotation process at the demonstration sites over the two-year evaluation period:

	Percent removal	
	Site I	Site II
BOD	50.1	60.4
TOC	47.1	50.4
Suspended solids	59.7	66.1
Volatile suspended solids	64.7	57.0
Total phosphorus	46.6	60.3

These values are the removals achieved during the entire two-year project. Most of 1973 was a period of startup and shakedown, therefore the 1973 results were generally lower than the average presented. The results obtained in 1974 are believed to be more representative of the efficiency of the screening/dissolved-air flotation process. The 1974 percent removals (concentration basis) were:

	Percent removal	
	Site I	Site II
BOD	57.5	65.4
TOC	51.2	64.7
Suspended solids	62.2	73.3
Volatile suspended solids	66.8	70.9
Total phosphorus	49.3	70.0

The results from Site II are better than Site I because the hydraulic loading was usually lower at Site II than at Site I resulting in lower overflow rates and longer tank detention times at Site II.

8. Calculation of the percent removals on a mass basis resulted in the following values:

	Percent removal	
	Site I	Site II
BOD	62.4	69.5
TOC	60.0	66.6
Suspended solids	67.6	69.8
Volatile suspended solids	73.6	67.3
Total phosphorus	53.2	62.4

The reason for the increase over the arithmetic means is that the overall treatment efficiency was usually better for long duration runs (large volumes treated) than for short duration runs (small volumes treated). Therefore, the mass removals are greater than the arithmetic mean which gives equal weight to each run without regard to the volumes treated.

9. From a mass balance the following estimates on sludge production for typical system operation were made:

	Site I	Site II
Duration of run, min	424	212
Volume of floated sludge, cu m (gal.)	21.4 (5,641)	106.4 (28,108)
Total sludge volume, cu m (gal.)	228.4 (60,343)	407.4 (107,635)
Suspended solids, %	0.64	1.29
Backwash water/total sludge volume, % of tot. sl. vol.	91	74
Volume of sludge produced/volume of overflow treated, cu m/1000 cu m	26.7	42.6

10. The floated sludge averaged 4.6 percent solids of which 36 percent was volatile matter.

11. Screen backwash water averaged 0.23 percent solids of which 60 percent was volatile matter. On the average, the backwash requirements were 2.7 percent of the plant flow.

12. No relationship could be established between the backwash water volumes used and the screen hydraulic and solids loadings.

13. When operating correctly, the chlorination system produced an effluent fecal coliform concentration of 113 colonies/100 ml. This number was below the standard of 200 colonies/100 ml set by the Wisconsin Department of Natural Resources for the Root River (standard for whole body contact).

14. Capital costs for the system can be expressed as follows:

	Site I	Site II	Site IIA
Total cost, $	436,599	841,420	25,001
$/cu m/day of treatment capacity	8.16	5.01	1.69
($/mgd of treatment capacity)	30,900	18,950	6,410
$/hectare of combined or storm sewer area	16,730	5,131	3,968
($/acre of combined or storm sewer area)	6,779	2,078	1,613

15. The operation and maintenance cost for the systems was 6.08¢/cu m (23.0¢/1,000 gal.). This cost probably could be reduced to 3.18¢/cu m (12.0¢/1,000 gal.) by process and procedural modifications (see Sect. II - RECOMMENDATIONS). The major reason for the high operation and maintenance cost is the cost of labor for maintenance of the sites and cleanup of the sites after a system operation. These costs were 3.94¢/cu m (14.9¢/1,000 gal.) or 65 percent of the total. Therefore, maintenance becomes the major cost item in the full-scale application of screening/dissolved-air flotation for the treatment of combined sewer overflows.

SECTION V, ROOT RIVER MONITORING STUDIES

16. The treatment of storm-generated discharges during this demonstration project has had a definite beneficial influence on the water quality of the Root River. The most noted change in the water quality of the River was a decrease in the fecal coliform concentrations when storm-generated discharges were treated as compared to when they were not treated.

17. Using the fecal coliform organism as an indication of river water quality at three points along the Root River for comparing wet weather and dry weather data over the entire demonstration period, the yearly geometric means listed below give an indication of quality changes which have occurred over time:

Fecal Coliform Concentration, No./100 ml

	1971 Dry	1971 Wet	1973 Wet	1974 Wet
Point A	353	5986	3084	1253
Point B	344	1775	2117	2057
Point C	84	265	860	813

Point A, nearest the river mouth is located in the immediate area of the treatment sites. Point B is upstream of Point A and Point C is upstream of both Points A and B. Of the three points monitored, only Point A showed a significant improvement in water quality during wet weather over the entire project. There was a 50 percent decrease in this parameter at Point A from 1971, when no treatment occurred, to 1973, when treatment started, and a further 50 percent decline from 1973 to 1974. This improvement is considered to be a direct effect of treatment on water quality.

Water quality at Point B measured by fecal coliform content, remained constant during the entire three-day monitoring period following a storm-generated discharge event. This site is located downstream of the last combined sewer overflow prior to the test reach.

Point C fecal coliform concentrations increased from 1971 to 1973 but remained unchanged from 1973 to 1974.

18. At each specific point there was improvement in water quality as indicated by dissolved oxygen levels over the duration of the monitoring program. Listed below are the mean dissolved oxygen concentrations for all three points over the three years monitored:

Dissolved Oxygen Concentration, mg/l

	1971 Wet	1973 Wet	1974 Wet
Point A	7.1	7.2	8.3
Point B	5.8	6.1	7.4
Point C	2.8	6.9	7.4

This general improvement in water quality cannot be solely attributed to the treatment system's operations because of other contributing factors affecting river DO.

19. During the entire period of monitoring (1971-1974) no change in the benthic deposits was noted in the areas of Points A and B. Point C,

located in a high energy area, was affected somewhat by scour during spring flooding. The benthic deposits at Points A and B, being relatively stable, represent a significant nutrient source and probably exert a high benthic oxygen demand.

20. The encroachment of Lake Michigan on the Root River had a significant influence on the water quality at the Points A and B monitoring areas. It affected most of the parameters monitored (dissolved oxygen, specific conductance, temperature) and made data interpretation difficult, if not impossible.

21. The selection of river monitoring sites, although placed at the best points available during this study, left much to be desired. There was, for instance, no monitoring site downstream of both treatment systems. The lack of such a point hindered the assessment of the effect of operation of the treatment systems on the quality of the river. In fact, all of the river monitoring locations were located upstream of the outfall of Site I and, therefore, no positive conclusions could be reached about the effect of this treatment unit on the river.

It would have been most beneficial, from the standpoint of monitoring water quality in the Root River, if the treatment plants could have been constructed just downstream of the Point C monitoring station. This would have allowed the placement of monitoring Points A and B downstream of the treatment plants but far enough upstream to eliminate the influence of Lake Michigan on the monitored area.

SECTION VI, STORM WATER MANAGEMENT MODEL

22. The Runoff and Transport blocks of the Storm Water Management Model (SWMM) have been shown to be adequate in predicting the quantity of arriving flow at the two combined sewer overflow treatment points, Sites I and II. These blocks have also been shown to be adequate in predicting the overall quality of the arriving flow at the two sites. The quality prediction was acceptable in terms of BOD and fecal coliform and fair in terms of suspended solids. The computed "first flush" concentrations for suspended solids lacked accuracy but the prediction of the remainder of the pollutograph was acceptable.

23. The Storage block was used (1) to verify the removals of pollutants through the treatment units by comparing computed and measured effluent concentrations, and (2) to design screening/dissolved-air flotation units for the remaining combined sewer overflows along the Root River. In both instances this block has yielded acceptable results.

24. The application of the SWMM to the modeled areas of this project produced manpower and cost requirements that may be summarized as follows:

a. Applying the SWMM to an urban drainage area requires one-man-day per 2 ha (5 acres) of drainage area to obtain sewer records, analyze them and produce the needed input data for the Runoff and Transport blocks.

b. Data debugging then requires one-man-day per 40 ha (100 acres) of simulation.

c. The costs in CPU time of running the Runoff and Transport blocks with 150 elements and 100 time steps averaged 60 seconds.

SECTION II

RECOMMENDATIONS

SECTION IV, TREATMENT SITES

1. Immediate action should be taken to treat or to otherwise abate the remaining volumes of overflow in the City of Racine. Satellite treatment plants employing the screening/dissolved-air flotation treatment process should be considered as a feasible alternative to combined sewer separation.

2. Large volumes of sludge will be generated throughout the City of Racine if satellite treatment facilities are employed to treat combined sewer overflows. If this sludge were bled back to the sewer system after the overflow subsided (as is done at the present demonstration systems), it would create an excessive load on the dry weather sewage treatment plant operations. Therefore, for future satellite plants it is recommended that sludge handling facilities be included. In addition, the volume of sludge could be significantly reduced by holding the screen backwash water and the floated sludge in separate tanks. At the demonstration systems the backwash water accounted for 80 to 90 percent of the sludge volume but averaged only 0.23 percent solids. Therefore, it is suitable for bleed-back to the sewer. The floated sludge, on the other hand, while accounting for only 10 to 20 percent of the sludge volume, averaged 4.6 percent solids. Therefore, if the floated sludge were collected separately, it might be feasible to treat it at the satellite plant and to prevent overloading of the dry weather plant.

3. Because capacity flow at Site II was infrequent, and because when capacity did occur, it was usually at the beginning of a run, and because Site I was usually hydraulically overloaded at the beginning of a run, a method of storage followed by the screening/dissolved-air flotation process may be beneficial at other sites in Racine, or in other cities. This use of storage would mean less initial hydraulic load on the system, especially the drum screens, and would reduce the required capacity of the system to handle a given storm and overflow event. This approach is basically a problem of optimizing design storage and treatment capacity.

4. The following recommendations are made regarding the equipment design:

    a. Use of an alternative source of water such as final effluent, river or city water for chemical dilution and screen backwash systems.

b. Greater structural support for the drum screen panels.

c. A new design for the drum seals.

d. Complete separation of the drum screen bypass channel for the drum screen chamber.

e. Inclusion of a method of removing accumulated solids from the drum screen chamber.

f. Use of a heavy-duty bar screen to eliminate the jamming of the rakes.

g. Use of air lines that will not deteriorate and are easily accessible.

h. Placement of flumes or other flow monitoring devices such that accurate flow measurements may be obtained.

i. An automated method of removing deposited solids from the bottom of the flotation tanks.

j. A different type of air controller for better control of pressurization-tank pressures.

Achievement of the design changes would significantly reduce the cost of treatment. It is estimated that the cost could be reduced from 6.08¢/cu m (23.0¢/1000 gal.) to 3.18¢/cu m (12.0¢/1000 gal.).

5. The following recommendations regarding operation are made to upgrade treatment efficiency at the sites:

   a. The bar screen rakes at Site II should operate continuously so that a buildup of material on the bar screen does not block the plant flow and cause bypass.

   b. The automatic startup equipment for the sites should be maintained so that the sites are always set to start automatically.

   c. When equipment problems occur, they should be corrected while operations are underway at the sites and during the overflow period, if possible. The sites should be shut down during an overflow, only if absolutely necessary.

   d. The sites should have the necessary personnel and chemicals available so they can be kept running until the combined sewer overflow has ceased.

6. As expected, no removals of dissolved pollutants were achieved by the screening/dissolved-air flotation process. If dissolved pollutant removal is required for upgrading the treated efficiency, an additional treatment step at Racine or for future installations will be needed.

7. Fifty percent removal of suspended solids from storm water was achieved by use of a 50 mesh screen. Increased removals may be possible if finer-mesh screening media is employed.

SECTION V, ROOT RIVER MONITORING STUDIES

8. Monitoring of the Root River during both wet and dry weather periods should be continued at the points monitored during this project. This period of monitoring should be carried on over at least the next three years, and should measure most of the same parameters measured during this program. This monitoring would build a substantial data base upon which to evaluate the water quality of the Root River as it passes through the City of Racine and to provide further input as to the impact of the treatment systems on the river. It would also provide:

   a. Comparison of dry weather water quality over a long term period to determine if there is a time related trend or change in the quality of the Root River.

   b. Comparison of dry weather to wet weather events in a given year to show change due to storm-generated discharges.

   c. Better comparison of storm events from year to year.

9. If possible, some method of analyzing water quality downstream of both Sites I and II should be devised and implemented to determine the total impact of the treatment systems on the receiving body.

10. For any future CSO demonstration project, if part of a project is to determine the effect of CSO treatment quality, the treatment sites should be located upstream of any oscillating influence such as Lake Michigan. Such a location would allow the effects of treatment on the water quality of the receiving body to be characterized more easily, if not more graphically.

11. Flow measurement devices should be installed or stage-flow relationships should be determined at various points in the test reach of the Root River.

12. Benthic productivity should be determined for each of the major areas of the Root River in the City of Racine. Analysis of the biomass and species of benthic organisms should also be performed. This research activity should be conducted during spring, summer, and fall and should only be done after a prolonged dry period. In each of the areas selected for benthic studies, a determination of the benthic oxygen demand should be made during each season.

13. In the event that future monitoring work on any type of system utilizes a constant recording/monitoring system as used in this project, it is strongly advised that the data collection systems be multiplexed in such a way that all data collected are compatible with computer systems.

The use of manual labor to reduce data from a strip chart is very costly and time consuming. A computer-compatible data system would allow the investigator more time to work on the relationships of different events.

14. A system approach to monitoring river-lake interaction areas should be developed. The Root River - Lake Michigan system, though somewhat estuarine in nature, must have a monitoring system which takes into account the subtle river - lake interactions. Such an approach would ensure that future monitoring efforts on the Root River could make fuller use of all data collected and also ensure that future monitoring efforts on similar systems could develop more meaningful field data.

SECTION VI, STORM WATER MANAGEMENT MODEL

15. A section in the User's Manual (38) is needed to provide information to calibrate the computed quantity and quality of the SWMM. This procedure would allow the user to monitor one overflow point and then calibrate the output so that predictions at other overflow points are accurate.

16. Expression of the coliform concentrations used in the SWMM should be at the user's option; either membrane filter counts or MPN.

17. The Storage block should provide more options to the user within each treatment device selected. These options should include variable chemical dosage rates, screen areas, and design flow rates to give better simulation at existing treatment units.

18. The evaluation of the other treatment options in the Storage block is needed using actual data from full-scale units.

19. The Receive block should be modified to accept smaller and less sophisticated receiving waters such as small rivers and streams. Better documentation, along with examples of actual application, is needed to provide the user with a basis to begin the Receive block application.

SECTION III

INTRODUCTION

III-1 COMBINED SEWER OVERFLOW PROBLEM

During recent years the discharge of raw untreated sewage as a result of combined sewer overflows (CSO) has become recognized as a serious pollution problem. As determined in a 1967 survey, approximately 29 percent of the total sewered population of the United States is served by combined sewers. Approximately three percent of the total annual sewage flow is discharged in the overflow which contains as much as 95 percent of the sewage produced during periods of rainfall (1).

The traditional solution to the problem is to provide separate sewer systems for storm and sanitary flows. For older, established areas of the city, such separation involves much expense and inconvenience. In addition, storm water runoff itself can be highly polluted (2)(3)(4)(5). For these reasons alternatives to sewer separation have been suggested, principally storage and/or treatment of the overflow.

There appear to be three alternative methods, or combination of methods, which can be utilized by a municipality to eliminate or minimize the pollution associated with CSO:

1. Construction of larger interceptors and expansion of dry weather treatment plant facilities.
2. Construction of holding tanks with provisions to pump the stored wastewater back into the system after the overflow subsides.
3. Treatment and discharge of overflows.

In those instances where the location of sewage treatment plants and existing interceptors make expansion economically attractive, the construction of larger interceptors and the required accompanying enlargement and/or modification of dry weather treatment facilities may provide a suitable solution to the combined sewer overflow problem. This approach has been successfully used in Kenosha, Wisconsin (6). However, in municipalities that have widely scattered overflows or where treatment facilities are not amenable to expansion due to process or area limitations, this approach does not appear to be economically feasible. Normal design capacity for interceptors is between 1.5 and 5.0 times the dry weather flow (7)(8). During a storm, the flow in a combined sewer may increase 50 to 100 times the dry weather flow (9). If a complex system of interceptors has to be enlarged to handle flows of this magnitude, or if the sewage treatment plant has to be relocated because the existing one cannot be modified to treat the total interceptor flow, construction costs may be prohibitive. Also, the problems of public inconvenience and lost business, caused by the construction required, must be considered.

The holding-tank concept has been and is being used as a method of handling overflows. The disadvantages of the method include the cost of the tank installation, the physical and economic limitations imposed by required holding capacities, and the need for returning the entire flow to interceptor systems for treatment after the storm subsides. In some locations, an "overloaded" condition exists at the treatment plant for several days following a major storm and the overflow would have to be retained in holding basins until the overloaded condition ceases. This delay would create health and odor control problems. In addition, any runoff occurring after holding tank capacity is reached, must be discharged untreated to receiving waters.

For these reasons, treatment and discharge of overflows near the point of the overflow has generated considerable interest. The Storm and Combined Sewer Section, Office of Research and Development, U. S. Environmental Protection Agency (EPA), has sponsored a number of research, development, and demonstration projects directed to establishing the feasibility of various processes suitable for "on-site" treatment of combined overflows. Studies conducted at Fort Smith, Arkansas showed that a system including a gyrating screen, hydrocyclones, and a total flow pressurization dissolved-air flotation unit effected an 84 percent removal of suspended solids and a 42 percent reduction of BOD (9). An 18,900 cu m/day (5 mgd) screening/dissolved-air flotation (sdaf) pilot plant operated by this contractor achieved suspended solids and BOD removals of from 70 to 80 percent during the highly pollutional "first flush" period (10). Based on this performance, the next logical step was to determine the technical and economic feasibility of utilizing the sdaf process for full-scale treatment of CSO.

In 1968 the Wisconsin Department of Natural Resources issued orders to the City of Racine, Wisconsin to reduce the pollution resulting from the discharge of raw overflow from the combined sewers in a 284 hectare (700 acre) area of the central city. The traditional approach at that time was to separate the combined sewers into storm and sanitary sewers. Estimated costs of sewer separation for the drainage area at the time were from $10 to $13 million, not including the substantial amount of inconvenience and lost business that would result from construction. It was estimated that a system of small satellite plants utilizing the sdaf treatment process could be installed at the major overflow points along this stretch of the river for approximately $4 million. Because such a system could save the City of Racine in excess of $6 million in construction costs and would be more effective from a pollution control standpoint than sewer separation, the City of Racine, assisted by the Environmental Sciences Division of Envirex Inc. submitted an application for a demonstration project to the U.S. Fedeal Water Pollution Control Administration (FWPCA). As conceived, the project was to establish the cost/performance criteria for the full-scale application of the sdaf process to treatment of combined sewer overflows as an alternative to sewer separation. In June, 1970 a grant offer was made by FWPCA to the City of Racine. In addition to the federal grant, commitments were made by the Wisconsin Department of Natural Resources and the City of Racine to provide additional funds.

III-2 PROJECT OBJECTIVES

The project had two objectives:

1. To evaluate the screening/dissolved-air flotation process developed under FWPCA Contract No. 14-12-40 as an alternative to the physical separation of those combined storm and sanitary sewers that overflow into the last 6.4 km (4 mi) of the Root River in Racine, Wisconsin.

2. To evaluate and modify (if required) the combined sewer mathematical model developed under FWPCA Contract 14-12-502, "Storm Water Pollution Control Management".

These overall objectives were expected to provide information on:

- The process adequacy of the treatment system as an alternative to separation of combined sewers in a 284 ha (702 acre) area of central Racine, Wisconsin.

- The cost/benefit relations to be expected from use of a treatment system as opposed to sewer separation in the subject area.

- Validity of FWPCA combined-sewer mathematical model for application to problems of any given area.

- Design, operation, and application criteria for the use of the sdaf treatment method as an alternative to combined sewer separation in any given area.

SECTION IV

TREATMENT SITES

IV-1 PRELIMINARY STUDIES

Evaluation of Candidate Reaches of the Root River

The selection of the project test reach was preceded by division of the Root River into a number of candidate reaches. Existing information and results of field observations, sampling and analytical determinations were used to make objective comparisons between and among the candidate reaches.

Through the use of sewer maps supplied by the City of Racine Engineering Department, the major drainage areas along the Root River and within the City of Racine were identified. These areas are identified by number in Figure 1. For the major area numbers, the total area in hectares was subdivided into the area served by combined sewers and the area served by separate storm and sanitary sewers (Table 1).

Initially, the Root River within the City of Racine was divided into four possible candidate study reaches as shown in Figure 1. A preliminary survey was made to determine if any of these candidate reaches were obviously unsuitable for further consideration. The decision was reached to eliminate Reach 4 for the following reasons:

1. Treatment at existing overflow locations in this reach would require construction of screening/flotation units in the backyards of a number of single family residences.

2. Within this reach, storm sewers serve 662.2 hectares (1636 acres) and combined sewers serve only 77.8 hectares (192 acres). Even by moving the reach upstream to eliminate strictly storm water discharges Nos. 35 and 36 (Figure 1) the separate sewered area would exceed the combined sewered area by a factor larger than 2 to 1. Within the work and budgeted scope of this project, it was felt that selection of this reach would result in a disproportionate expenditure of project funds for treatment of storm water only.

3. The branch in the river at Island Park would complicate the river monitoring program because of the division of flow in the two channels.

The judgment reached from these three considerations was to reject this candidate reach and give it no further consideration for this demonstration project.

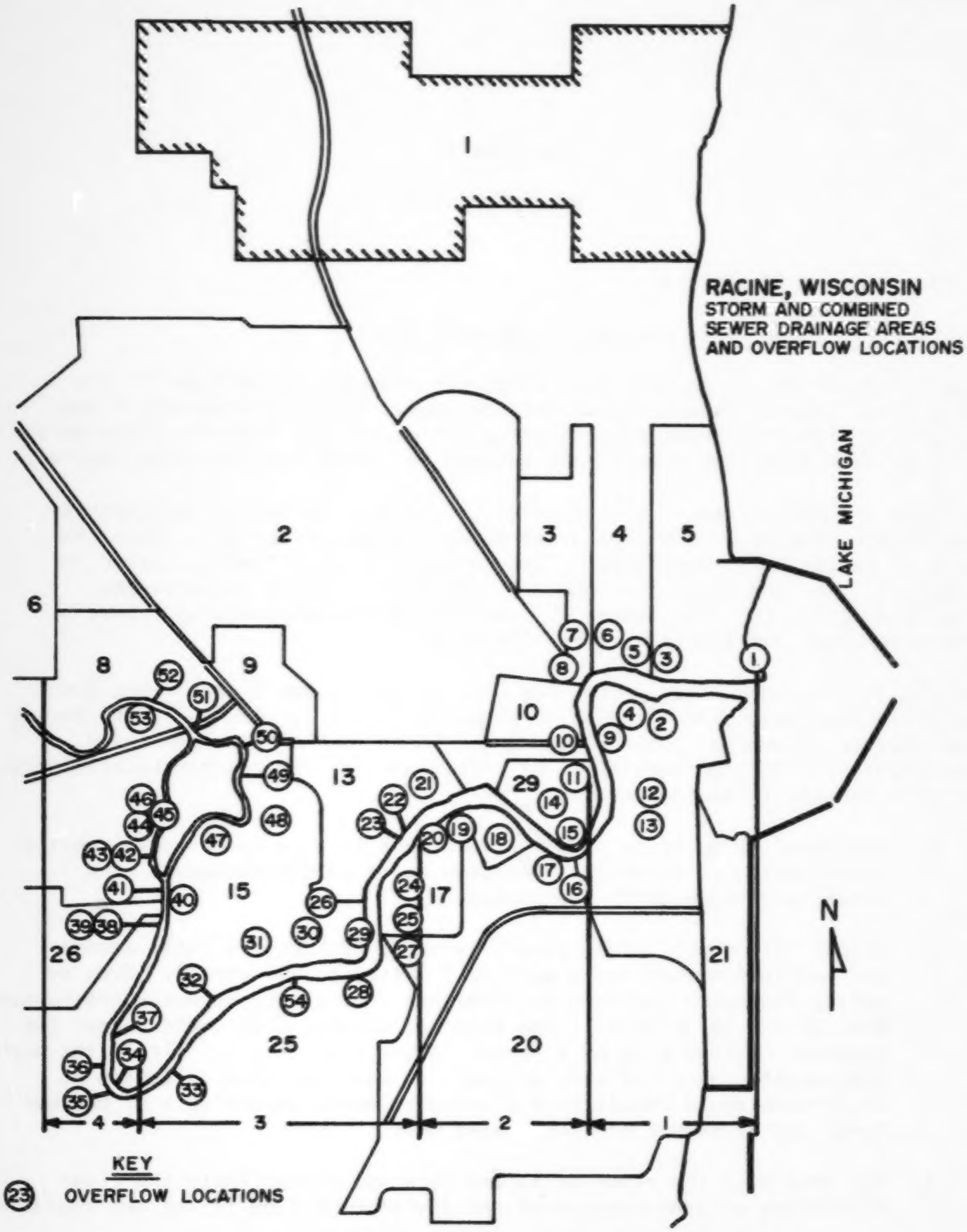


Figure 1. Drainage areas, combined and storm sewer overflows, and division of river for selection of project test reach.

TABLE 1. AREAS SERVED BY COMBINED SEWERS AND SEPARATE SEWERS AS OF 1970

Area No.	Total area		Combined sewers		Separate sewers	
	Hectares	(acres)	Hectares	(acres)	Hectares	(acres)
1	712.8	1760.0	144.6	357.0	568.2	1404.0
2	211.4	522.0	131.2	324.0	80.2	198.0
3	17.2	42.5	17.2	42.5	0.0	0.0
4	19.0	46.9	19.0	46.9	0.0	0.0
5	29.6	73.2	14.4	35.6	0.0	0.0
6	26.1	64.5	0.0	0.0	26.1	64.5
7	2.96	7.32	0.0	0.0	2.96	7.32
8	14.8	36.6	14.8	36.6	0.0	0.0
9	14.8	36.6	0.0	0.0	14.8	36.6
10	16.8	41.6	16.8	41.6	0.0	0.0
11	5.3	13.2	0.0	0.0	5.3	13.2
12	29.8	72.6	24.7	61.0	0.0	0.0
13	23.7	58.6	14.8	36.6	8.9	22.0
14	2.37	5.86	0.0	0.0	2.37	5.86
15	18.4	45.4	8.3	20.4	10.1	25.0
16	2.7	6.6	2.7	6.6	0.0	0.0
17	11.9	29.3	0.0	0.0	11.9	29.4
18	2.37	5.86	2.37	5.86	0.0	0.0
19	1.9	4.7	1.9	4.7	0.0	0.0
20	116.2	287.0	24.8	61.2	91.4	225.8
21	8.9	21.9	0.0	0.0	8.9	21.9
22	3.0	7.33	0.0	0.0	3.0	7.33
23	3.6	8.8	3.6	8.8	0.0	0.0
24	5.9	14.65	5.93	14.65	0.0	0.0
25	68.2	168.5	0.0	0.0	68.2	168.5
26	44.6	110.0	0.0	0.0	44.6	110.0
27	502.2	1240.0	0.0	0.0	502.2	1240.98
28	1.0	2.46	0.4	1.0	0.59	1.46
29	105.0	26.0	10.5	26.0	0.0	0.0
30	127.6	315.0	48.6	120.0	79.0	195.0

Preliminary Mathematical River Modeling - Because of the complexity of the drainage systems and of the response of the Root River to the inputs of storm water runoff and combined sewer overflow, a modeling consultant (Hydroscience, Inc., Leonia, New Jersey) was retained to assist in the reach selection process. The objective of the river modeling was to determine which discharges should be treated to yield maximum water quality benefits to the river.

The following information and procedures were used in the storm discharge and river quality model:

1. The combined sewer and storm water outfalls were simulated as shown in Figure 2.

2. Drainage areas, runoff coefficients, dry weather flows and sewer capacities used are tabulated in Table 2.

3. Daily averages were used for dry weather flow (DWF); the program made no distinction as to time of day in which the storm event occurred.

4. Removal efficiencies through treatment devices were as follows:

    BOD treated = 0.4 (BOD applied)
    SS treated = 25 + 0.06 (SS applied)

5. The program simplified discharge quality variation by assuming either of two conditions to exist:

    a. First Flush Quality Conditions - The highest concentration of contaminants associated with initial flushing conditions are assumed to be present the entire first hour of runoff.

       Exception - If the antecedent storm ended 12 hours or less prior to the start of the storm under study, the program assumes that "first flush conditions" do not exist, and that concentrations associated with sustained discharge quality will occur in the first hour, as well.

    b. Sustained Discharge Quality - The lower concentrations of contaminants resulting from dilution with rainwater are assumed to prevail from hour No. 2 throughout the duration of the discharge event.

6. Table 3 summarizes the quality parameters in the program. In the opinion of the consultant, there is reasonably good documentation for using the values and for the use of a 12-hr period to define "flush first" conditions. The values were considered reasonable for use in this preliminary evaluation.

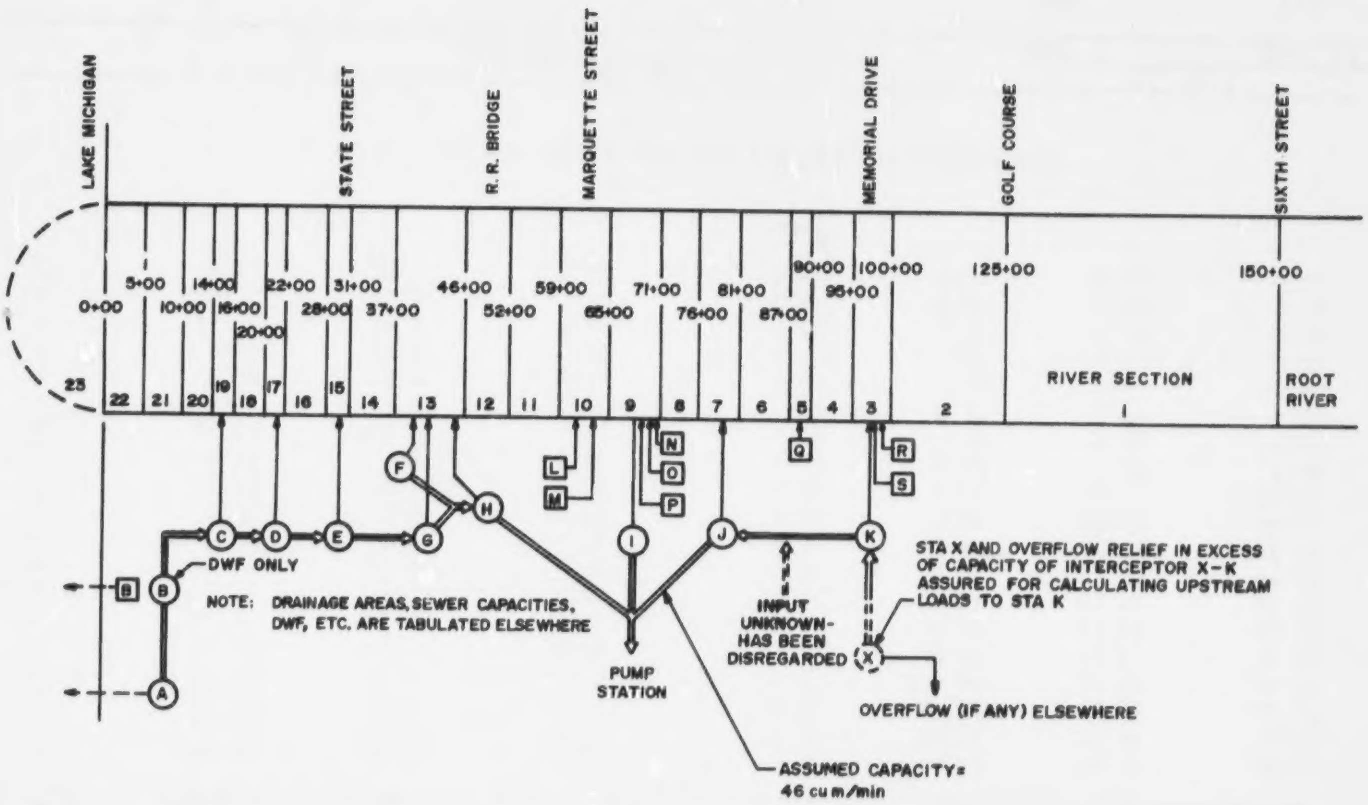


Figure 2. Schematic diagram of simulation used in preliminary river model.

Table 2. INPUT CONDITIONS FOR PRELIMINARY RIVER MODEL

Station	Area		Runoff coefficients	Dry weather flow		Interceptor capacity	
	hectares	acres		cu m/min	cfs	cu m/min	cfs
A	144.59	357.0	0.45	7.94	4.67	64.77	38.10
C	50.63	125.0	0.53	3.74	2.21	25.84	15.20
D	106.92	264.0	0.53	7.89	4.64	35.53	20.90
E	22.28	55.0	0.75	0.15	0.09	42.67	25.10
F	12.56	31.0	0.75	0.58	0.34	42.67	25.10
G	24.71	61.0	0.75	0.34	0.26	42.67	25.10
H	2.03	5.0	0.60	0.02	0.01	42.67	25.10
X	15.31	37.8	0.45	4.22	2.48	28.39	16.70
K	1.90	4.7	0.45	0.03	0.02	28.39	16.70
J	0.00	0.0	0.00	0.03	0.20	28.39	16.70
I	23.73	58.7	0.45	0.39	0.23	42.67	25.10
L	3.08	7.6	0.80	0.00	0.00	0.00	0.00
M	65.61	162.0	0.53	0.00	0.00	0.00	0.00
N	8.51	21.0	0.70	0.00	0.00	0.00	0.00
O	1.90	4.7	0.50	0.00	0.00	0.00	0.00
P	5.47	13.5	0.50	0.00	0.00	0.00	0.00
Q	15.39	38.0	0.49	0.00	0.00	0.00	0.00
R	43.74	108.0	0.48	0.00	0.00	0.00	0.00
S	4.17	10.3	0.45	0.00	0.00	0.00	0.00

Table 3. QUALITY VALUES USED FOR PRELIMINARY RIVER MODEL

	BOD, mg/l		SS, mg/l	
	Hour 1	Hour 2+	Hour 1	Hour 2+
Untreated overflow	175	40	600	150
Overflow after treatment	70	16	61	34

7. To permit inclusion of stormwater runoff in the preliminary model, it was assumed that stormwater runoff is similar in quality to combined sewer overflows with respect to BOD and suspended solids, the parameters analyzed in this preliminary evaluation.

8. Treatment efficiencies were assumed to apply equally for all discharges. Treatment units were assumed to be able to handle all flows at full efficiency.

9. The river water was assumed to have no BOD. Dissolved oxygen was assumed to be at saturation as the river enters the test section. The river quality model thus shows only the impact of the storm generated discharges.

10. River discharge rates obviously increase in response to storms. Data from the test period and superficial analyses of a few other periods indicated a lag of a day or two before the full effect is felt in the test area. These flows will be reduced in quality due to runoff contamination upstream from the test area, although the preliminary model assumed no contamination to be present.

11. Rainfall records used were those for the Milwaukee Airport Weather Bureau Station. Annual rainfall distribution and storm patterns in the project area are expected to be similar.

Three years of actual rainfall data were selected to obtain simulated storm generated discharges and associated quantities of BOD and suspended solids for locations indicated in Table 4. The three years selected included a wet year (1960), an average year (1961) and a dry year (1963). This information, summarized in Table 4, showed that overflow locations, C, D, L, and M were responsible for the major portion of the total pollution loads discharged into the entire reach.

The time variable model used to predict water quality was used to simulate the river conditions during a 200-hr period in August 1965. The results of this analysis have been plotted in Figure 3 for four critical stations. The critical section of the river based on BOD and DO considerations in Section 9 upstream from sections closer to the large combined discharges from positions C, D, and E. The influence of dispersion near the mouth of the river, other lake-influenced causes, low river velocities, and the large ratio of overflow to river discharge rate were considered responsible for this effect. The predicted location of minimum DO was confirmed by a survey conducted on September 23, 1970. It did not appear, however, that the magnitude of the sag would be sufficient to reduce the DO below 5.0 mg/l which was established as the minimum permissible value in the State of Wisconsin Water Quality Standards (11).

The model could not be readily adapted to handle coliform data. Therefore, it was not possible to model this parameter. However, on October 28, 1970, during a storm generated discharge event, a number of river samples from various locations were collected and analyzed for fecal coliform. The results of this survey are presented in Figure 4.

TABLE 4. SIMULATED OVERFLOW VOLUMES AND CHARACTERISTICS FOR RACINE, WISCONSIN

Combined sewer outfall	K	--	J	I	--	F,G,H	E	D	C
Storm sewer outfall	R,S	Q	--	N,O,P	L,M	--	--	--	--
1960 - 208 storms									
Rainfall - 103.41 cm (40.71 in.)									
Overflow volume, cu m	238,455	75,700	0	98,410	382,285	242,241	60,560	480,695	450,415
m gal.	63	20	0	26	101	64	16	127	119
No. times overflow occurred	630	630	13	630	630	274	105	322	274
Total BOD mass, kg	13,691	4,446	12	5,750	21,953	13,767	2,870	27,142	25,632
lb	30,189	9,793	27	12,666	48,356	30,324	6,322	59,785	56,459
BOD mass after treatment kg	5,476	1,778	5	2,299	8,781	5,506	1,329	10,856	10,252
lb	12,063	3,917	11	5,066	19,342	12,129	2,929	23,914	22,583
Total SS mass, kg	49,632	16,112	46	20,846	79,559	49,992	12,140	98,506	92,966
lb	109,442	35,489	102	45,917	175,235	110,116	26,742	216,975	204,773
SS mass after treatment, kg	8,966	2,903	9	3,768	14,338	9,150	2,316	17,958	16,866
lb	19,749	6,396	19	8,301	31,583	20,156	5,103	39,556	37,151
1964 - 161 storms									
Rainfall - 71.58 cm (28.18 in.)									
Overflow volume, cu m	166,540	52,990	0	68,130	261,165	162,755	45,420	325,510	295,230
m gal.	44	14	0	18	69	43	12	86	78
No. times overflow occurred	444	444	9	444	444	168	66	220	168
Toall BOD mass, kg	11,403	3,633	13	4,864	17,940	11,414	3,552	22,384	19,520
lb	25,118	8,003	28	10,714	39,517	25,141	7,824	49,305	42,996
BOD mass after treatment, kg	4,561	1,453	5	1,945	7,176	4,565	1,420	8,953	7,807
lb	10,047	3,201	11	4,285	15,807	10,056	3,129	19,722	17,198
Total SS mass, kg	40,794	13,005	46	17,378	64,217	40,819	12,629	80,093	70,015
lb	89,856	28,647	101	38,279	141,449	89,911	27,818	176,418	154,220
SS mass after treatment, kg	6,622	2,121	7	2,788	10,474	6,607	1,940	13,023	11,621
lb	14,586	4,672	16	6,141	23,071	14,555	4,275	28,686	25,599
1963 - 144 storms									
Rainfall - 48.51 cm (19.10 in.)									
Overflow volume, cu m	.110	.034	0	.045	.178	.098	.022	.197	.185
m gal.	29	9	0	12	47	26	6	52	49
No. times overflow occurred	425	425	6	425	425	122	45	157	124
Total BOD mass, kg	6,710	2,199	7	2,811	10,860	5,964	1,421	11,908	10,880
lb	14,780	4,844	15	6,192	23,921	13,137	3,129	26,230	23,964
BOD mass after treatment, kg	2,684	879	3	1,125	4,344	2,386	568	4,763	4,351
lb	5,912	1,937	6	2,477	9,568	5,255	1,251	10,492	9,585
Total SS mass, kg	24,222	7,937	26	10,146	39,193	21,557	5,118	43,013	39,409
lb	53,411	17,483	55	22,347	86,329	47,534	11,274	94,743	86,803
SS mass after treatment, kg	4,234	1,385	4	1,770	6,840	3,807	881	7,559	7,077
lb	9,326	3,051	9	3,899	15,065	8,386	1,941	16,650	15,588

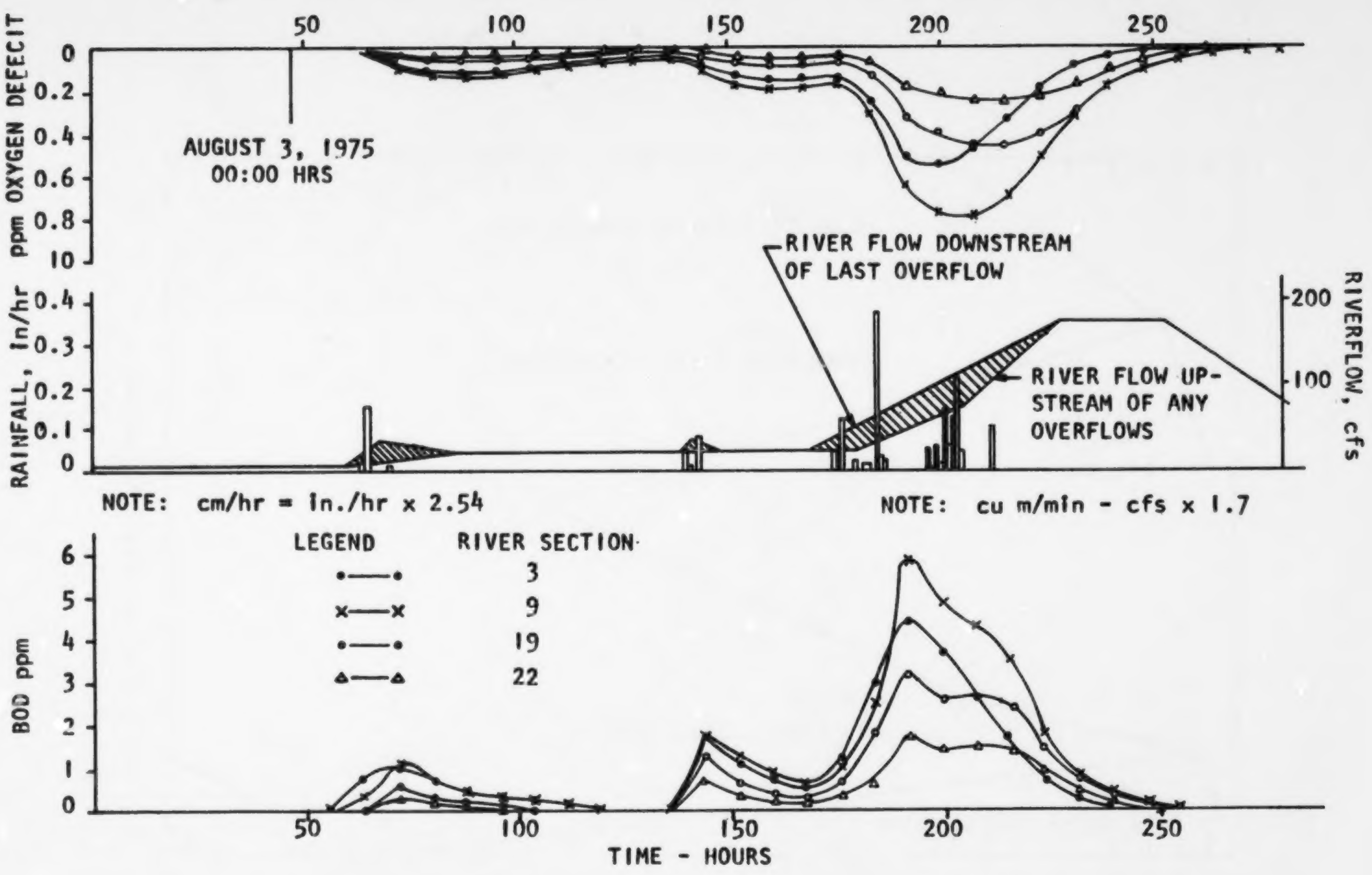


Figure 3. Effect of combined and stormwater overflows on the Root River in Racine, Wisconsin, as predicted by the preliminary river model.

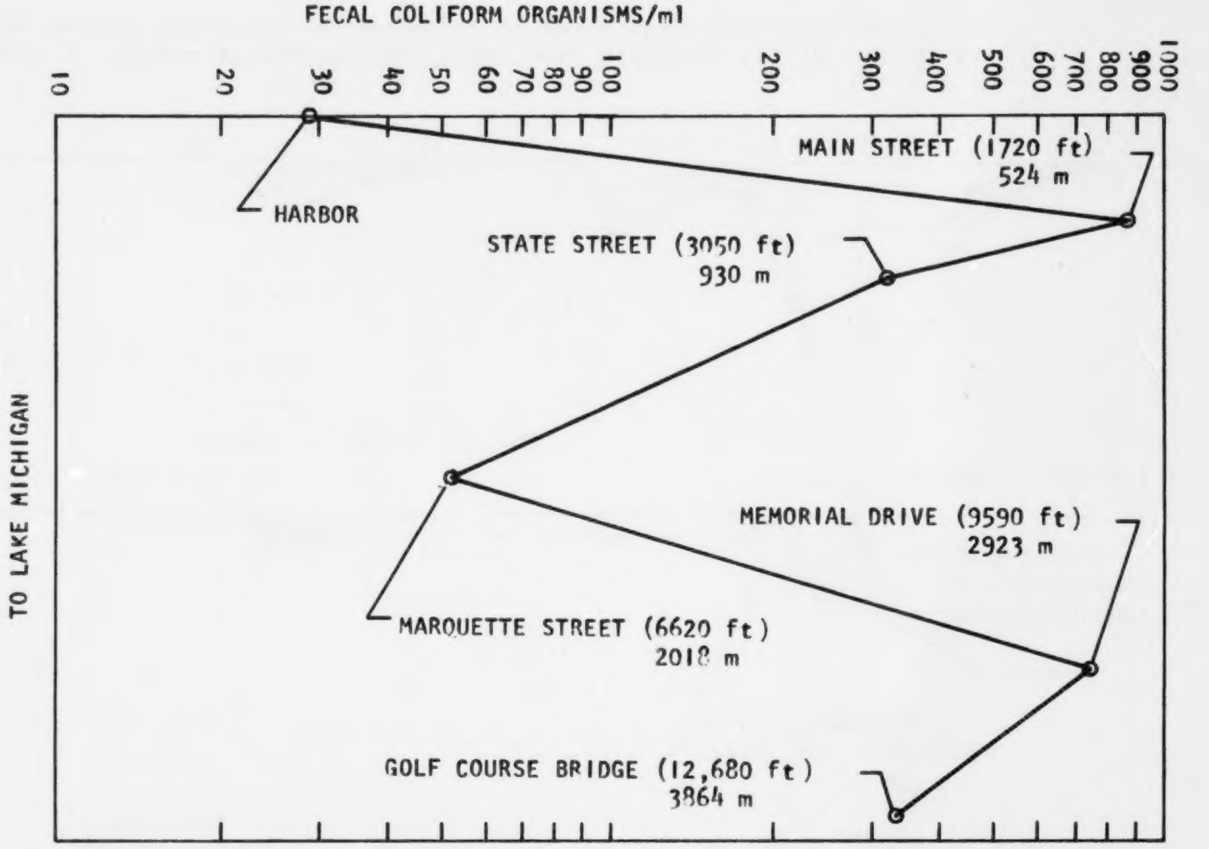


Figure 4. Fecal coliform concentrations at Root River sections, October 28, 1970.

Fecal coliform counts at all sampling positions exceeded the State of Wisconsin Standard for partial body contact. The high coliform counts at Memorial Drive and the municipal golf course bridge probably resulted from large volumes of combined sewage overflowing into the Root River at overflow Nos. 35 and 36 (Figure 1). At that time separation was in progress in areas draining to these overflow locations and was expected to be completed before May, 1971. A large reduction in coliform counts was expected when separation was completed.

Fecal coliform counts at Main and State Streets appear to be adversely affected by combined sewage from overflow Nos. 1, 3, 7, and 8 (Figure 2). This region of the Root River was used intensively by recreational boats operating from facilities along the Root River and in the inner harbor. Unintentional whole body contact with water in this region of the Root River was an occasional experience by employees of the marinas and individuals operating boats on the river. Personal communications with such individuals confirmed subjectively the objective evidence of poor water quality in this reach.

Based on the mathematical model estimates of quantities of BOD and suspended solids discharged to the Root River, treatment of discharges in the lower reach of the river would provide the maximum water quality benefit to the river and harbor area.

The results of the modeling effort and the water quality survey were summarized as follows:

- The minimum dissolved oxygen concentration occurs in Section 9. However, this sag, under average flow conditions, is not expected to reduce the dissolved oxygen concentration of the river below the recommended standard of 5.0 mg/l.

- The fecal coliform concentration in the lower reach of the river is greatly in excess of the State of Wisconsin water quality standards for partial body contact.

- Protection of Lake Michigan and the recreational uses of the navigable portion of the river indicates treatment sites should be located in the lower reach.

Additional Considerations - Factors that have an influence upon reach and treatment site selection include land availability and cost, feasibility of construction and relative cost of construction at alternative sites, and for each site, the effect of water quality. Nine alternative combinations of sites were evaluated objectively. The locations of these sites are shown in Figure 5.

The criteria used for making objective comparisons are shown in Table 5. Numerical values on a scale of 0 to 4 (least favorable - 0, most favorable - 4) were assigned to land availability and land cost at each site. These two factors together with the design discharge rates and BOD removal estimates were then transformed to a scale of 0 to 10 (each number was multiplied by 2.4

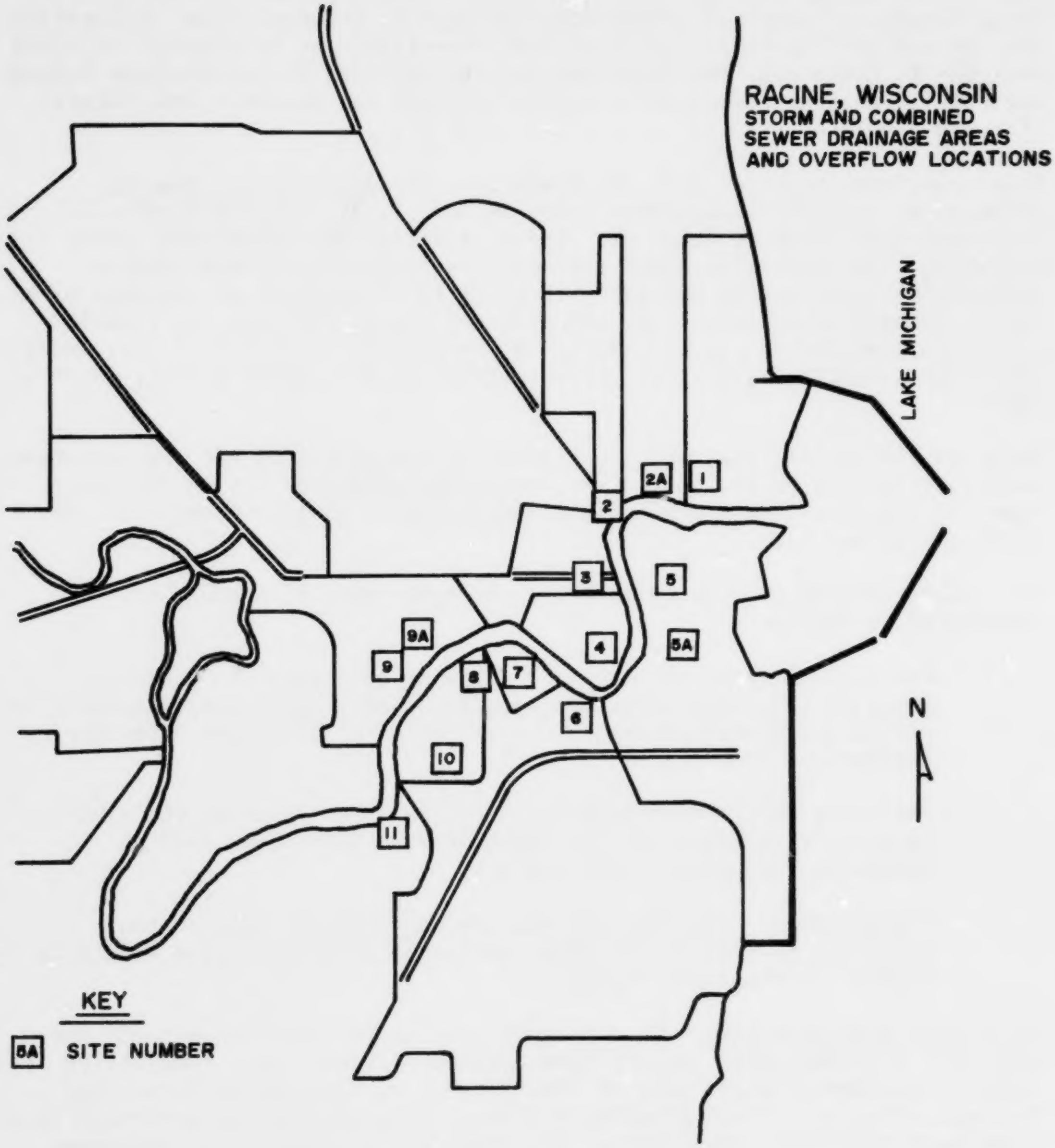


Figure 5. Alternative sites for CSO treatment plants.

TABLE 5. EVALUATION OF ALTERNATIVE SITES

(1) Alter-nate No.	(2) Sites[a]	(3) Sewer overflow number	(4)	(5) Design Flow[b]				(6) Minimum River[c] Distance in reach		(7) BOD[d] removed		(8) Land avail-ability	(9) Land cost	(10) Points	(11) Treat-ment Cost, $
		Storm	Combined	Storm		Combined									
				cu m/day	mgd	cu m/day	mgd	km	mi	kg	lb				
1	I, II, IIA	5	1,3,6,7,8	11,400	3	253,600	67	0.64	0.40	>291	>640	3.7	3.7	37.7	$821,000
2	II, III, V	--	7,8,9,10	0	0	283,900	75	0.43	0.27	<272	<600	3.3	3.3	35.9	$858,000
3	II, IIA	5	6,7,8	11,400	3	177,900	47	0.18	0.11	>104	>230	4.0	4.0	30.1	$565,000
4	II, IIA, III	5	6,7,8,10	11,400	3	230,900	61	0.40	0.25	>127	>280	4.0	4.0	32.8	$739,000
5	III, V, VA, IV, VI, VII	12	9,10,15,16 17,18,13	11,400	3	155,200	41	0.55	0.34	<148	<325	2.7	2.7	24.3	$729,000
6	IV, VA, VI, VII, VII, IXA	12,19,20 21,22	15,16,17	280,000	74	49,200	13	0.61	0.38	<141	<310	2.5	2.3	25.2	$749,000
7	VII, VII, IXA, IX	19,20,21 22	18,23	268,700	71	90,800	24	0.61	0.38	<87	<191	2.3	2.0	23.5	$731,000
8	VII, IXA, IX, X	19,20,21, 22	23,24,25, 26	268,700	71	102,200	27	0.58	0.36	<87	<191	2.0	1.8	22.4	$730,000
9	IX, IXA, X, XI	21, 22, 28	23,24,25,26	68,100	18	102,220	27	0.64	0.40	<31	< 69	1.8	1.8	15.3	$505,000

a. Location shown in Figure 5; sewer overflows shown in Figure 1.

b. Based on a precipitation rate of 1.17 cm/hr (0.46 in./hr) and antecedent precipitation of 1.17 cm (0.46 in.) total during the preceding 5 hours.

c. Distance of river affected by elimination of combined sewer overflows.

d. Assuming 60% BOD removal.

to transform the value to a point scale ranging from 0 to 10). Discharge rate and BOD removal estimates for each site were assigned "points" on a scale from 0 to 10. The value "10" was assigned to the site having the largest value for each parameter. Other sites were assigned points on this scale in direct proportion to the magnitude of each parameter with respect to the site having the maximum value. A weighting factor was used to compute points for the discharge rate to give somewhat more weight to combined overflows and less to storm water only.

Point values were obtained for combinations of sites listed in Column 2, Table 5, and totals were tabulated in Column 10. Sites were combined into alternatives listed in Column 1 by consideration of the total estimated cost for installations at each site and funds budgeted for construction of treatment facilities. The treatment cost (Column 11) is the total estimated cost of constructing screening/dissolved-air flotation systems at the various sites for each alternative. These estimates were used as a guide to select the project test reach.

Three additional factors taken into consideration in selection of the test reach were:

1. Overflow No. 2, 4, 9, and 13 are primarily the result of a severely undersized interceptor sewer. This sewer is being relaid and increased in size to eliminate overflows No. 2, 4, and 9. When reconstruction of this sewer is completed, there will be no overflow on the south side of the river from 4th Street, east of the lake.

2. Overflow No. 14 has been eliminated by plugging the discharge end of the pipe.

3. With a treatment system located at Site II, sufficient flow can be diverted from the interceptor so that no overflows occur at overflow No. 10, 11 and 15 for storms of intensity 1.3 cm/hr (0.5 in./hr) or less.

Selection of Project Test Reach - Evaluation of the river modeling and water quality survey, the objective evaluation table (Table 5) and the above special considerations lead to the conclusion that the test reach of river should be from Lake Michigan upstream to overflows No. 7 and 8. It was concluded that overflows in this reach should be treated by screening/dissolved-air flotation systems at Sites I and II and by a single screen for storm water at Site IIA (Figure 5).

The preliminary studies, based on a precipitation rate of 1.17 cm/hr (0.46 in./hr) and antecedent precipitation of 1.17 cm (0.46 in.) total during the preceding 5 hr, predicted flow rates of 123.5 cu m/min (47 mgd) at Site II, 52.6 cu m/min (20 mgd) at Site I, and 7.89 cu m/min (3 mgd) at Site IIA (Table 5). Due to land restrictions, Sites I and II could not be built large enough to handle these flows. Using all the available space at the sites resulted in capacities of 37.1 cu m/min (14.1 mgd) at Site I and 116.7 cu m/min (44.4 mgd) at Site II. The capacity of Site IIA is 10.3 cu m/min (3.9 mgd).

The minimum length of river in the selected test reach is 0.6 km (0.4 mi). For storms of 1.3 cm/hr (0.5 in./hr) or less, without considering any antecedent precipitation as was done for Table 5, the plant capacities were expected to prevent any untreated discharge between Lake Michigan and Ontario Streets, a distance of approximately 1.37 river km (0.85 river mi).

Pretreatment Storm Generated Discharge Studies

With selection of the project test reach, combined overflows No. 1, 3, 6, 7, and 8 and storm sewer discharge No. 5 were to be treated (see Figure 1). Overflows No. 1 and 3 were to be treated at the site east of Main Street (Site I) and overflows No. 5, 7 and 8 at the combined site west of Main Street (Site II/IIA). Little or no overflow occurred at overflow No. 6 and it was decided it would be possible to bulkhead the 20 cm (8 in.) discharge sewer.

During 1971 a study program was conducted to determine the quality and quantity characteristics of the discharge at the selected discharge points.

Description of Overflow Mechanisms - Overflow No. 1 located at Michigan and Dodge Streets and overflow No. 3 located at Chatham and Dodge Streets are separate and distinct overflow points. They were grouped together for contributing combined sewer area because the Michigan and Dodge Streets outfall serves as a relief overflow for the Chatham and Dodge Streets combined sewer overflow area.

The combined sewer overflow mechanism at Michigan and Dodge Streets is shown in Figure 6. It consists of a simple 30.5 cm (12 in.) high concrete weir in a 91.4 cm (36 in.) interceptor sewer just downstream from the 30.5 cm (12 in.) concrete relief interceptor which carries normal dry weather flow west on Dodge Street. The Chatham and Dodge Streets overflow (No. 3) is the relief for a 137.1 cm (54 in.) interceptor flowing south on Chatham Street and into the 91.4 cm (36 in.) outfall sewer (Figure 7).

Overflow No. 5 has no retention mechanism as it is only a storm water collection system servicing a 6.3 ha (15.5 acre) area. Storm water enters the last manhole in the sequence through a 20.3 cm (8 in.) sewer and flows into a 30.5 cm (12 in.) line serving as the discharge sewer to the Root River (Figure 8).

Overflows No. 7 and 8 combined into a single discharge chamber and flow to the river through two side-by-side 167.6 cm (66 in.) outfalls. A sketch of overflows No. 7 and 8 is presented in Figure 9. Two separate combined sewer interceptors enter the overflow box. One is a 228.6 cm (90 in.) trunk sewer which serves the west and central portions of the combined sewer area and the other is a 91.4 cm (36 in.) interceptor which serves the central area. Flow entering the chamber drops into a 99.1 cm (39 in.) interceptor flowing west on Dodge Street by means of 70.0 cm (24 in.) orifices as shown in Figure 9.

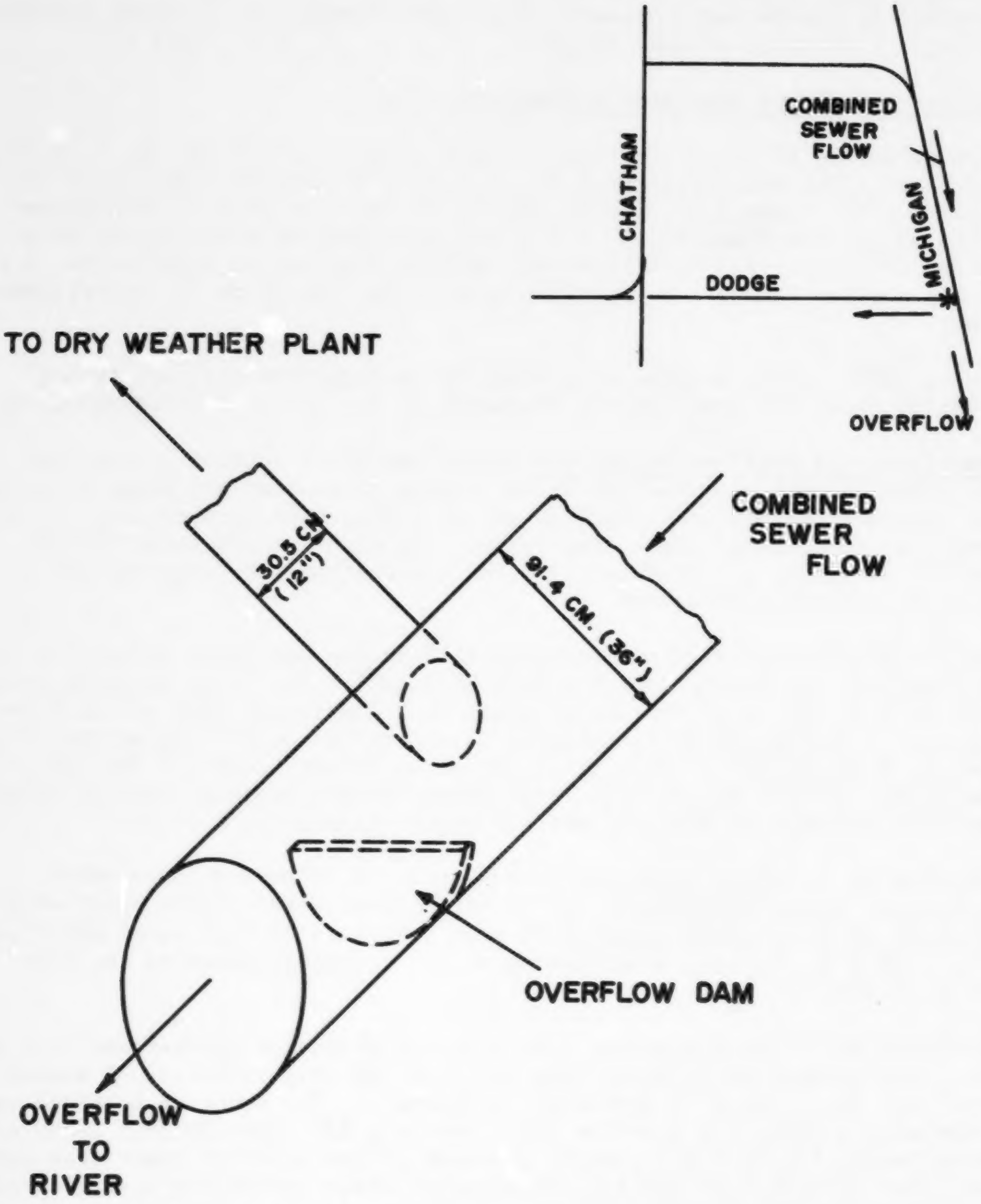


Figure 6. Michigan and Dodge Streets overflow mechanism, Discharge No. 1.

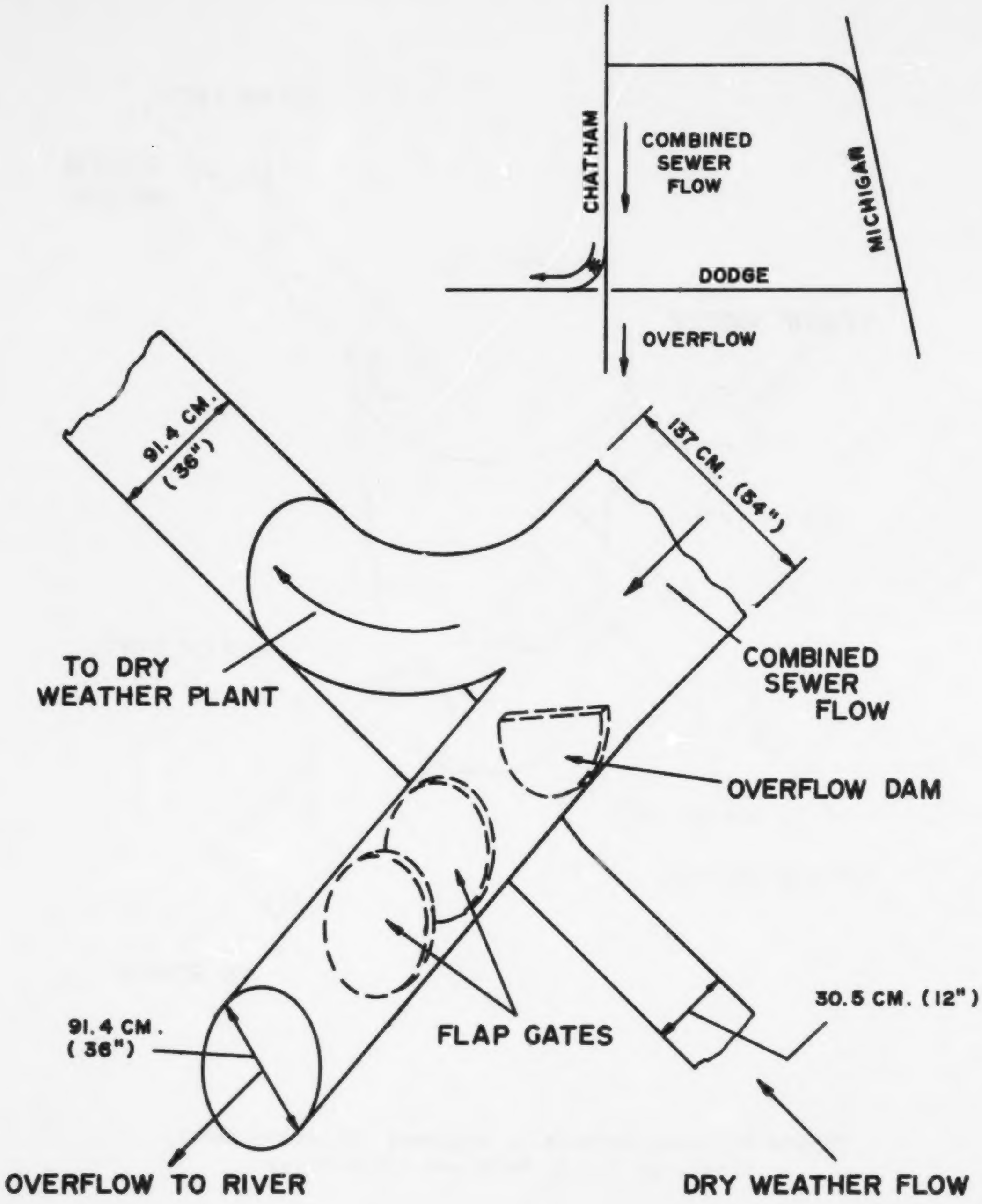


Figure 7. Chatham and Dodge Streets overflow mechanism, Discharge No. 3.

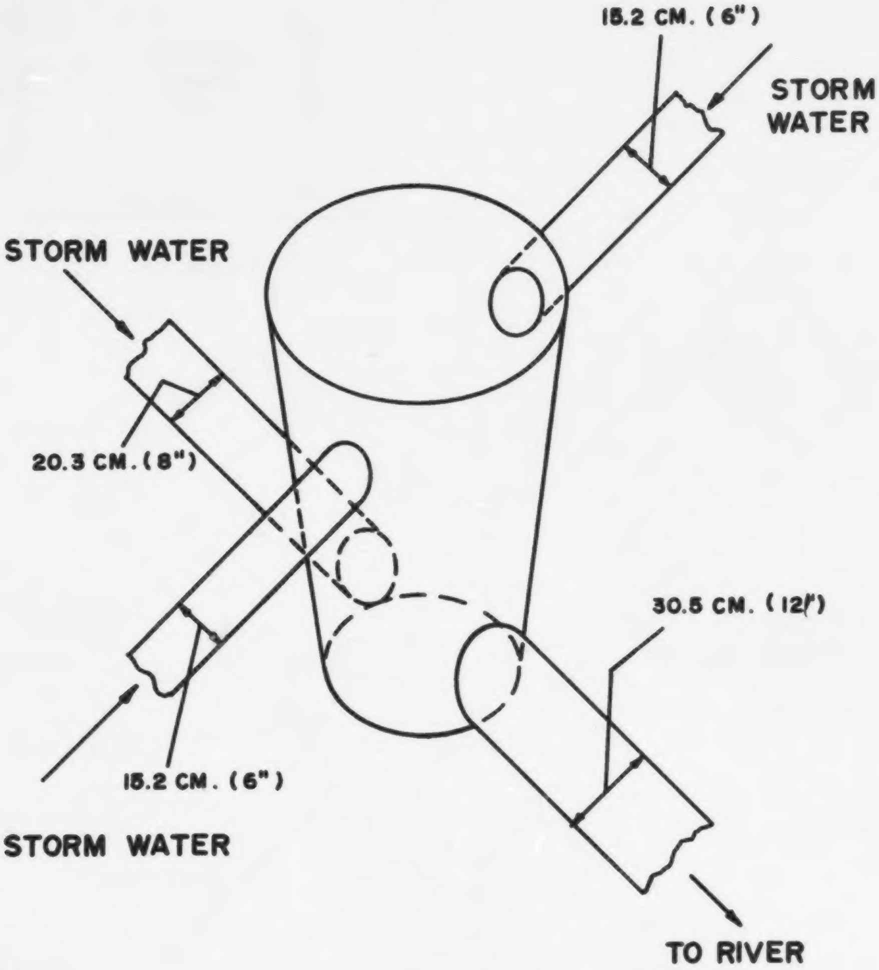


Figure 8. Last manhole in sequence for storm sewer, Discharge No. 5, Main and Dodge Streets.

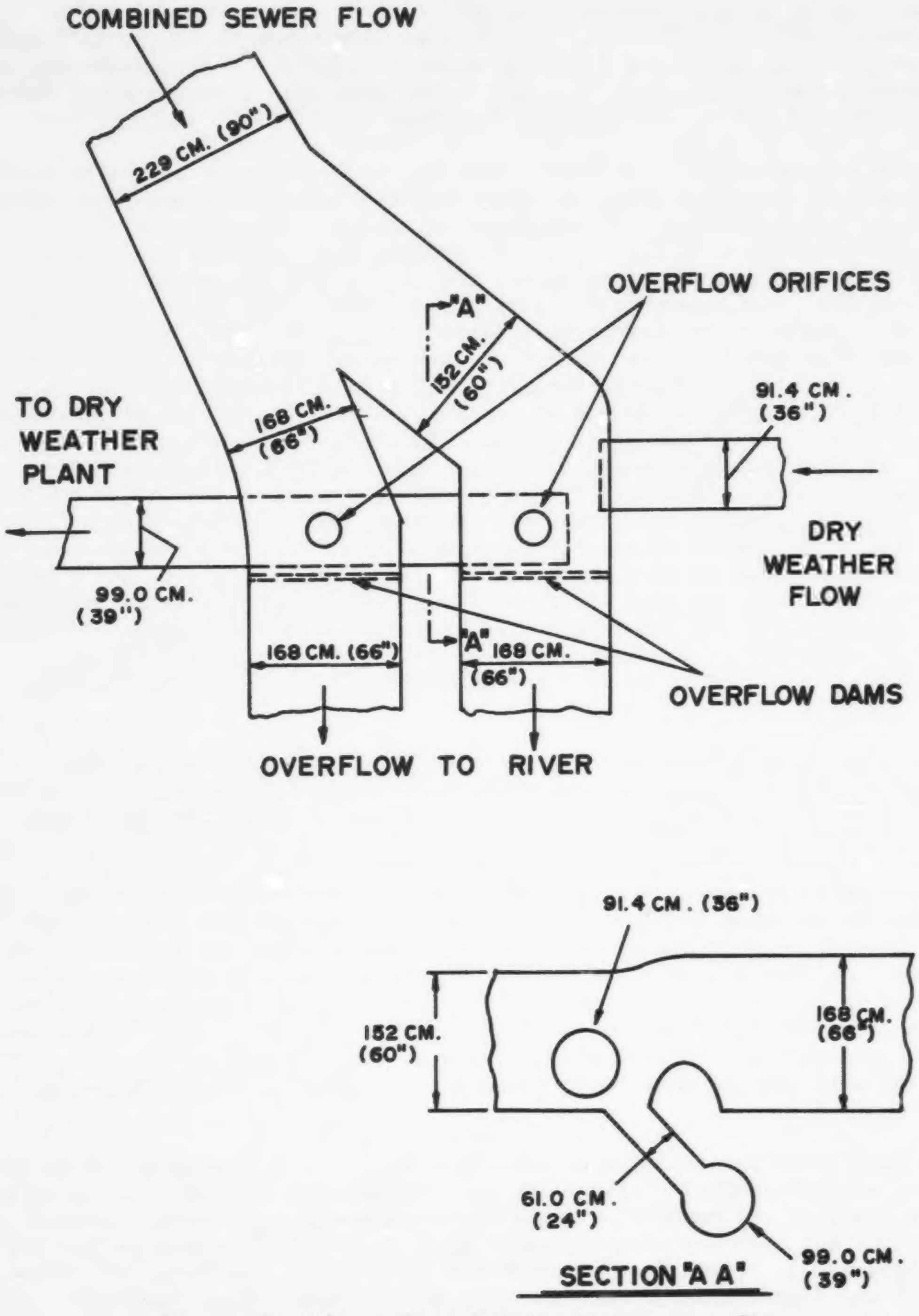


Figure 9. Wisconsin and Dodge Streets overflow mechanism, Discharge No. 7 and 8.

Flow Measurement and Sampling Program - Automatic sampling and depth recording instrumentation was installed at discharges No. 1, 3, 5, 7 and 8. Since overflows No. 7 and 8 entered a common chamber before discharge, one composite sample was drawn from the mixing zone each time the sewer sampler was activated. Flow was measured for these overflows at a common weir.

Sampling occurred at each location automatically during a discharge event. The samplers were specially designed for this application and were later to be used for sampling of the treatment processes. The interior of the sampler is shown in Figure 10 and the sample program cycle is shown in Figure 11. Each time before a sample is taken, the sample line is purged. During the preliminary studies automatic startup and shutdown was controlled by a remote switch located in the discharge chamber (Figure 12). Samples are collected in one liter bottles to a maximum of 24 discrete samples. Sample interval and the indexing of the sampling distributor arm is controlled by a timer which allows a time variation on sampling interval selection of from 1 sample every 5 min to 1 sample every 60 min. The sample sequence control for all four of the discharge samplers was set at a ten minute interval. This allowed for the collection of 24 discrete samples in a 4-hr discharge period. The discrete samples collected for discharges No. 1, 3, and 5 were composited by flow using the discharge record at each site. At overflows No. 7 and 8 the discrete samples were collected for individual analysis. Based upon the duration of the discharge event, discrete samples were isolated every 10 min for the first 30 min and at carefully selected time intervals thereafter for individual analysis characterizing the discharge event. All collected samples were placed in coolers for transportation back to the laboratory for analysis.

Discharge rates and volumes were originally to be determined at each location with a float-type liquid level recording instrument. Due to turbulent flow during discharge events, this method of measurment was found unsatisfactory. An attempt was made to control turbulence using a stilling well but with limited success. Depth recording instruments operating on a differential pressure principle were then procured. These instruments were installed as shown in Figure 13. By means of an aquarium pump, ambient air was introduced into tubing which ran from the recorder to the dam in the sewer. As flow (head) in the sewer increased, pressure built up within the tubing in increasing increments. These pressure increases were converted to depth readings and logged on circular charts. The chart was divided into 24 equal segments and rotated electrically at a one-cycle-per-day rate. The charts were changed once every three days or after every discharge, whichever came first.

The depth recorder employed at overflows No. 7 and 8 also operated on the basis of differential pressure but was mechanically dissimilar. Differential pressure was measured using a servo-manometer, converted to depth reading, and recorded with a punched tape recorder. The samples for discharges No. 1, 3, and 5 were composited by flow for analysis. Overflows No. 7 and 8 were sampled discretely at predetermined time intervals. At overflows No. 7 and 8, when the samples were taken discretely versus time, the laboratory data were used to calculate a composite value utilizing the

SAMPLER
ARM

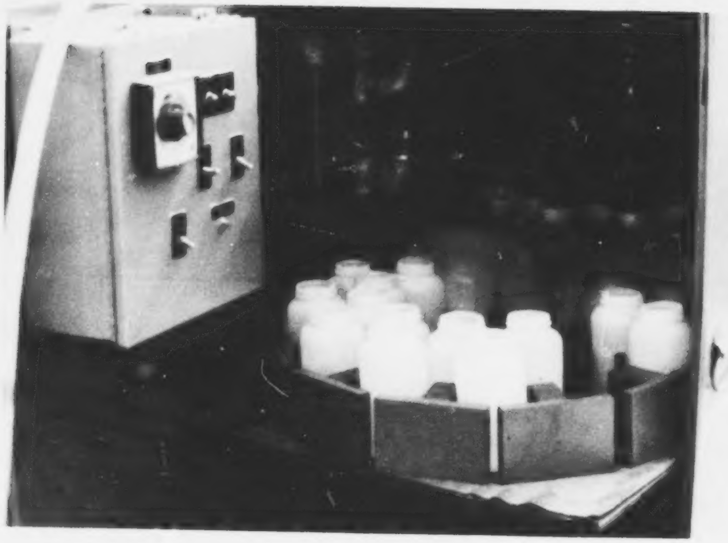


Figure 10. Inside of discrete sampler.

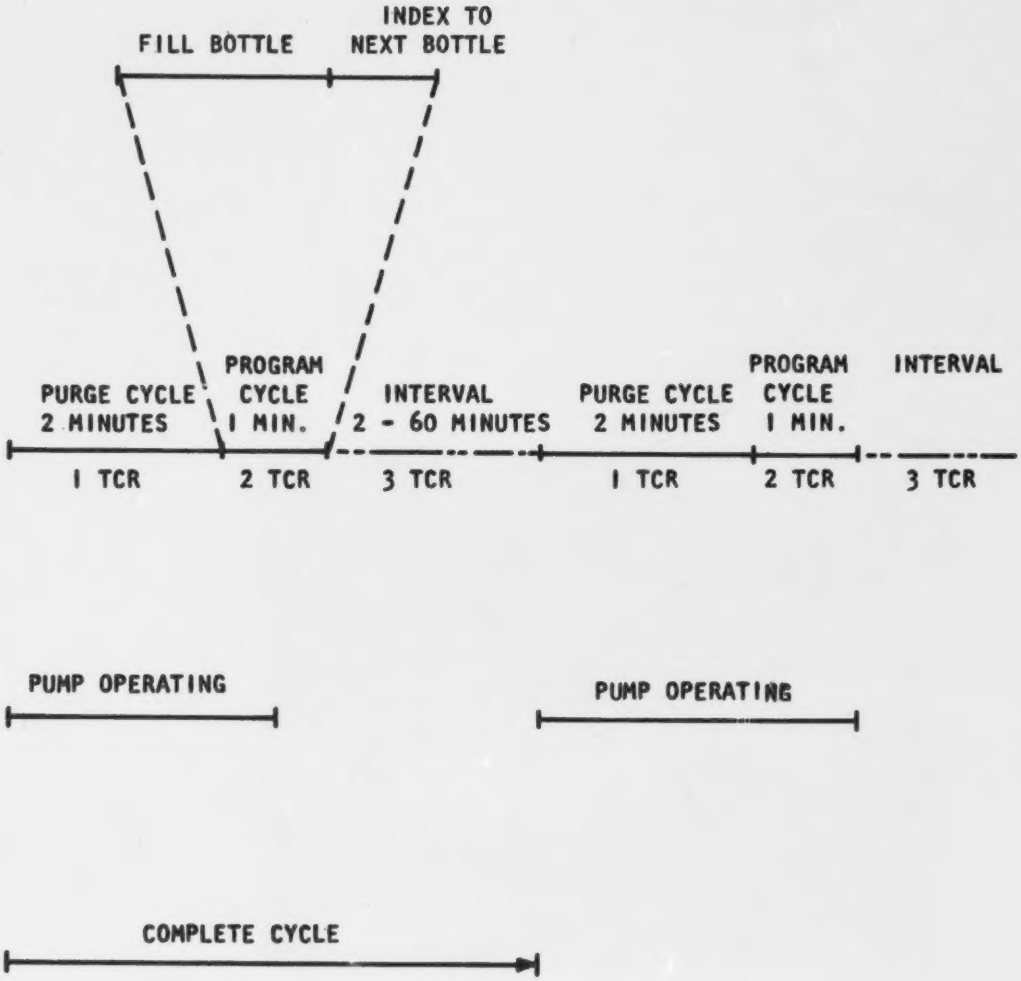


Figure 11. Program cycle for automatic sampler.

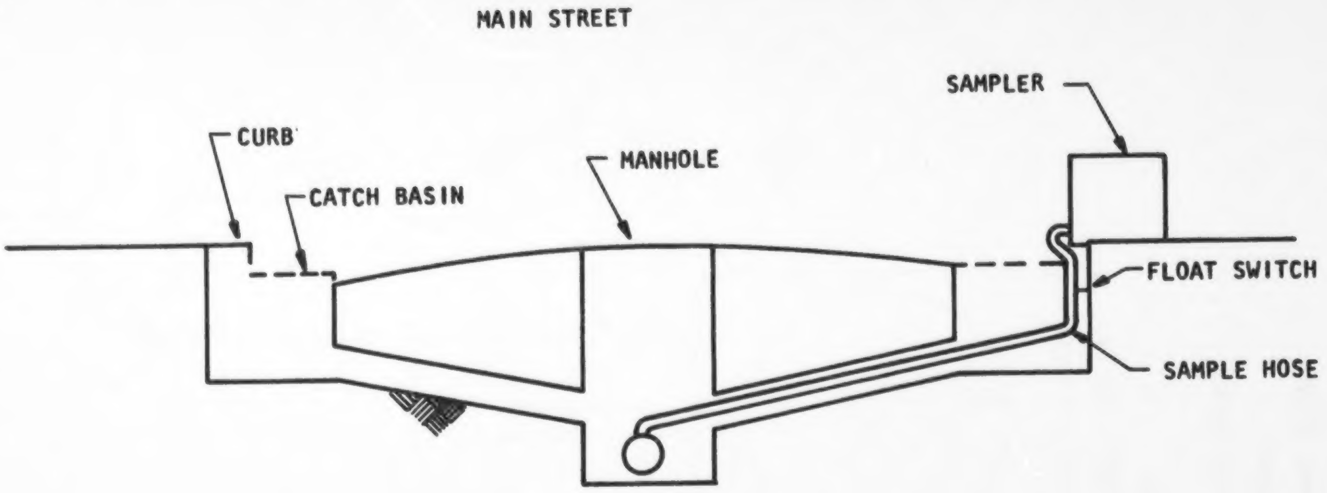


Figure 12. Typical sampler installation (installation at overflow No. 5, in storm sewer).

1. BELLOWS ASSEMBLY (PRESSURE CONVERTOR).
2. DEPTH RECORDING PEN AND CHART.
3. PRESSURE LINE TO BELLOWS.
4. CONTINUOUS GAS PRESSURE SOURCE (AQUARIAN PUMP).
5. PRESSURE LINE TO SEWER.
6. INSTALLATION OF PRESSURE LINE ON WEIR IN SEWER.

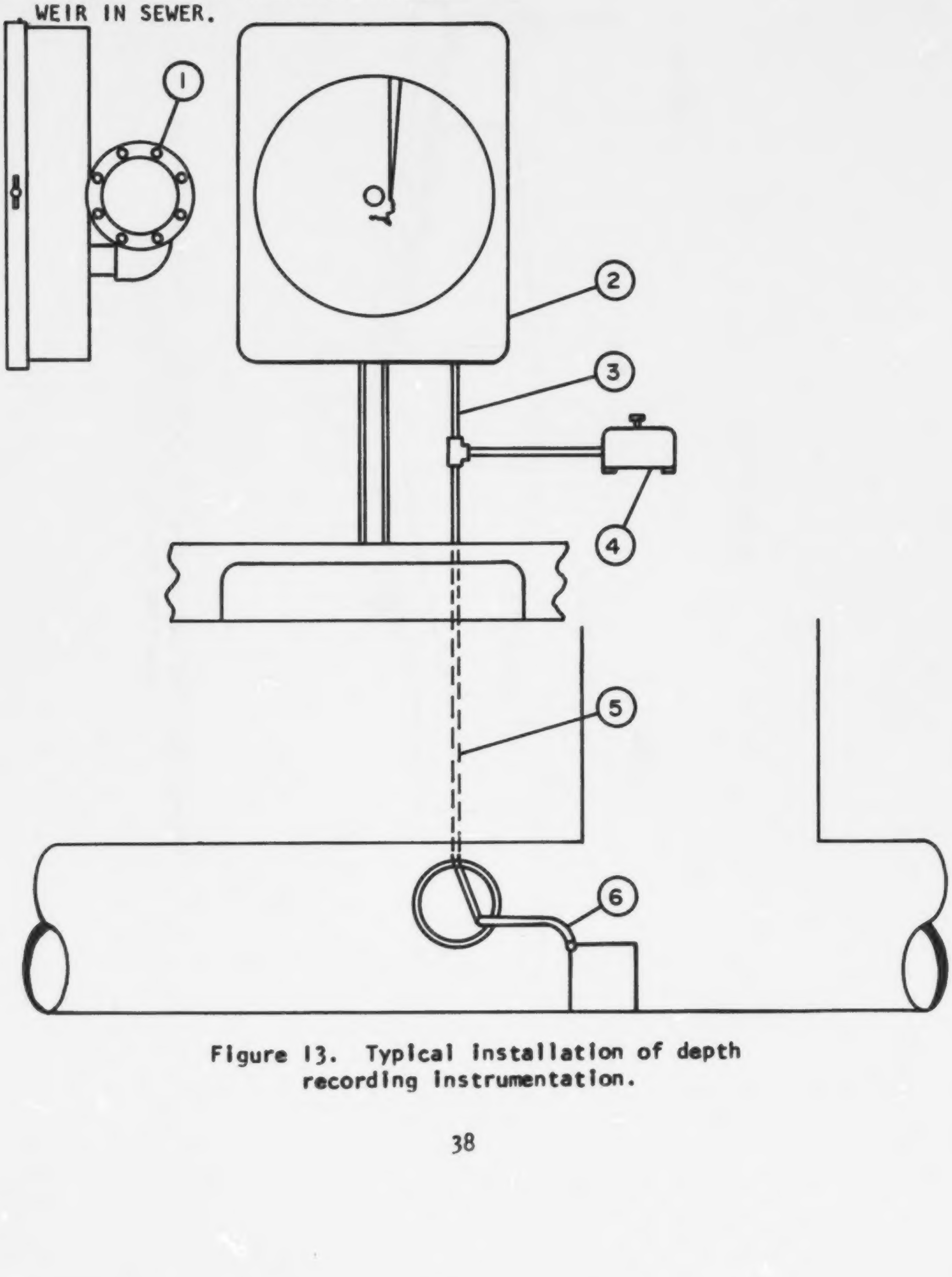


Figure 13. Typical installation of depth recording instrumentation.

flows recorded for each corresponding overflow. The average discharge characteristics for 1971 for discharges No. 1 and 3, 5, and 7 and 8 are presented in Tables 6, 7, and 8, respectively. These values are compared to the 1973 and 1974 discharge characteristics later in this section to determine if there was a significant difference between the 1971, and 1973-1974 data. In addition, averages were calculated for all discrete samples collected at overflows No. 7 and 8; they are presented in Table 9 on a quality versus time basis (minutes after start of overflow). The most concentrated discharge was at overflow No. 3 followed closely by No. 1. Discharge on a time basis shows the first flush occurring within 10 minutes followed by a general decrease in concentration. The raw data from the discharge quality analyses is presented in tabular form in Appendix IV-A, Tables A1 to A10.

Table 10 presents a record of discharge volumes as recorded for each discharge location. The turbulence difficulties discussed previously account for the absence of some volume determinations apparent in the table. The discharge volume at overflows No. 7 and 8 (Site II) was for the most part equal to or greater than the combined volume for overflows No. 1 and 3 (Site I). In addition, the overflow rate was usually considerably greater for overflows No. 7 and 8, although the duration of overflow was shorter than for overflows No. 1 and 3. These measurements substantiated the need for more treatment capacity at Site II than at Site I.

IV-2 SYSTEM DESIGN AND CONSTRUCTION

Description of Sites

The sites chosen for the location of the treatment plantsare shown in Figure 14 at Sites I, II and IIA. Sites I and II are screening/dissolved-air flotation treatment facilities for the treatment of combined sewer overflow. Site IIA, adjacent to Site II, consists of a single rotating drum screen for the treatment of storm water. The contributing areas for each site are given in Table II and are also shown in Figure 14. To some extent the contributing areas for Sites I and II overlap due to the interconnection of sewers as shown in Figure 14. The contributing areas for each site given in Table II indicate the site to which the largest volume of combined sewer overflow discharges during an average hypothetical rainfall occurrence. Also, these numbers represent only areas which overflow directly to the site. Upstream areas which may contribute to the overflow through surcharge in an interceptor have not been included if there is an upstream overflow point.

Site I, located on the bank of the Root River approximately one block east of the Main Street bridge, is shown in Figure 15. Sites II and IIA are located directly west of the Main Street bridge on the same parcel of land and also on the bank of the river. Sites II and IIA are shown in Figure 16. All three sites are located on the fringe of the downtown area and close to the mouth of the river.

Fortunately, sufficient land, already owned by the City of Racine, was vacant close to the sites of the outfalls. The availability of land was

TABLE 6. 1971 OVERFLOW CHARACTERISTICS - SITE I
OVERFLOWS NO. 1 AND 3

Michigan and Dodge Street overflow (No. 1)

Parameter	Mean[a] concentration, mg/l	Range	No. of events
BOD	79	38 - 152	10
TOC	98	24 - 203	9
Total solids	521	218 -1062	11
Suspended solids	298	38 - 596	10
Dissolved solids	236	102 - 563	11
Orthophosphate (as P)	0.64	0 - 1.20	11
Fecal Coliform density, No/100 ml	10,300	2,000 - 1,300,000	9

Chatham and Dodge Street overflow (No. 3)

Parameter	Mean[a] concentration, mg/l	Range	No. events
BOD	212	140 - 406	9
TOC	194	110 - 340	9
Total solids	943	560 -1649	10
Suspended solids	669	366 -1506	9
Dissolved solids	314	230 - 544	10
Orthophosphate (as P)	2.07	1.10 - 3.10	10
Fecal Coliform density, No./100 ml	21,900	1000 - 2,170,000	9

a. Means given are arithmetic except for Fecal Coliform density which is geometric.

TABLE 7. 1971 OVERFLOW CHARACTERISTICS - SITE IIA
OVERFLOW NO. 5

Parameter	Mean[a] concentration, mg/l	Range	No. of events
BOD	39	12 - 29	6
TOC	51	36 - 84	5
Total solids	608	381 -1012	6
Suspended solids	445	64 - 801	6
Dissolved solids	140	19 - 317	6
Orthophosphate (as P)	0.08	0 - 0.28	6
Fecal Coliform density, No./ml	23	1 - 417	5

a. Means given are arithmetic except for Fecal Coliform density which is geometric.

TABLE 8. 1971 OVERFLOW CHARACTERISTICS - SITE II
OVERFLOW NO. 7-8

Parameter	Mean[a] concentration, mg/l	Range	No. of events
BOD	212	48 - 282	9
TOC	238	36 - 184	9
Total solids	646	280 -1348	11
Suspended solids	669	181 - 847	9
Dissolved solids	274	100 - 501	11
Orthophosphate (as P)	0.75	0.21 - 3.17	11
Fecal Coliform density, No./100 ml	10100	540 - 143,000	9

a. Means given are arithmetic except for Fecal Coliform density which is geometric.

TABLE 9. OVERFLOW QUALITY VS. TIME, OVERFLOWS NO. 7-8 (SITE 11), JUNE 18 - NOVEMBER 18, 1971, ARITHMETIC MEANS FOR 10 OCCURRENCES

Parameter	Units	10 min	20 min	30 min	60 min	90 min	120 min	240 min
Fecal coli	colonies/ 100 ml	73375	87406	54349	55816	28862	15263	62300
$O\text{-}PO_4$	mg/l	1.33	1.39	1.96	1.19	2.17	1.18	.88
BOD	mg/l	170	117	97	82	92	48	60
TOC	mg/l	120	81	108	84	71	49	54
Total solids	mg/l	1107	969	775	627	545	476	525
Suspended solids	mg/l	843	369	472	360	207	185	232
Dissolved solids	mg/l	264	327	283	267	338	291	293

TABLE 10. 1971 OVERFLOW VOLUME DATA

Date	Rainfall		Average Intensity		Overflow No.1		Overflow No.3		Overflow No.5		Overflow No.7-8	
	cm	in	cm/hr	in/hr	cu m	x10^3 gal	cu m	x10^3 gal	cu m	x10^3 gal	cu m	x10^3 gal
7/23/71	0.86	0.34	0.74	0.29	---	---	---	---	---	---	213.0	56.27
8/2/71	0.74	0.29	0.25	0.10	---	---	---	---	---	---	290.1	76.64
8/10/71	2.16	0.85	0.71	0.28	---	---	---	---	---	---	11,320	2990
8/22/71	1.32	0.52	2.64	1.04	2,020	534	---	---	156.0	41.3	1,883	497.6
9/20/71	1.73	0.68	0.41	0.16	2,430	643	---	---	67.8	17.9	1,995	527.2
9/27/71	1.02	0.40	7.10	2.40	1,000	265	---	---	643	17.0	2,840	750.0
10/13/71	0.56	0.22	0.20	0.08	266	70.2	2,490	659	---	---	993.6	262.5
11/1/71	2.26	0.89	1.70	0.67	3,970	1050	1,940	512	245	64.8	23,830	6296
11/18/71	0.30	0.12	0.23	0.09	386	102	194	51.2	---	---	2,719	718.4

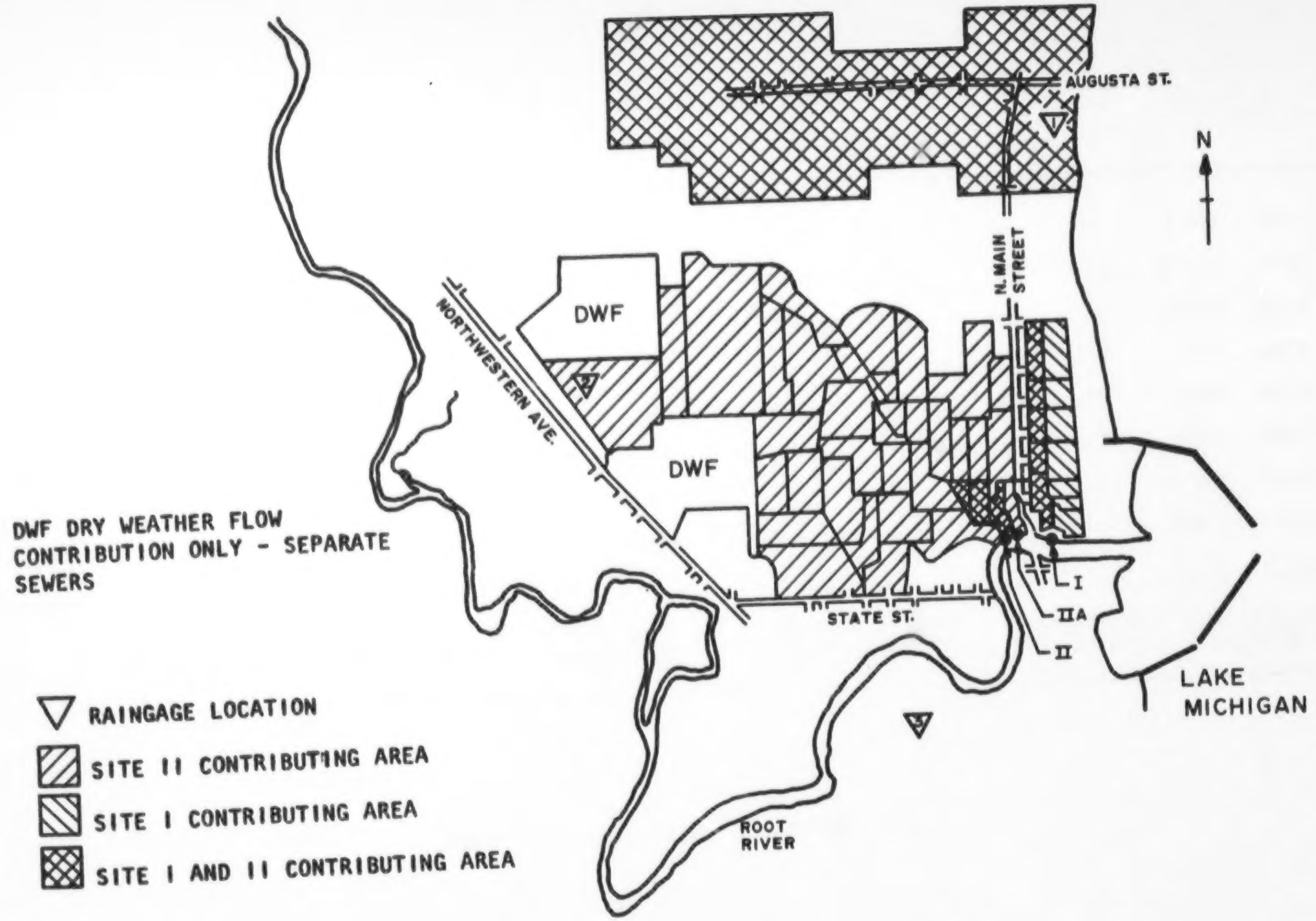

AUGUSTA ST.
N
N. MAIN STREET
NORTHWESTERN AVE.
DWF
DWF
STATE ST.
I
IIA
II
LAKE MICHIGAN
ROOT RIVER
1
2
3
DWF DRY WEATHER FLOW CONTRIBUTION ONLY - SEPARATE SEWERS
▽ RAINGAGE LOCATION
▨ SITE II CONTRIBUTING AREA
▧ SITE I CONTRIBUTING AREA
▩ SITE I AND II CONTRIBUTING AREA

Figure 14. Rain gage and treatment site locations and contributing areas.

FRONT WEST

MAIN STREET
BRIDGE STATION

CONTROL BUILDING

FRONT EAST

Figure 15. Site I treatment facility.

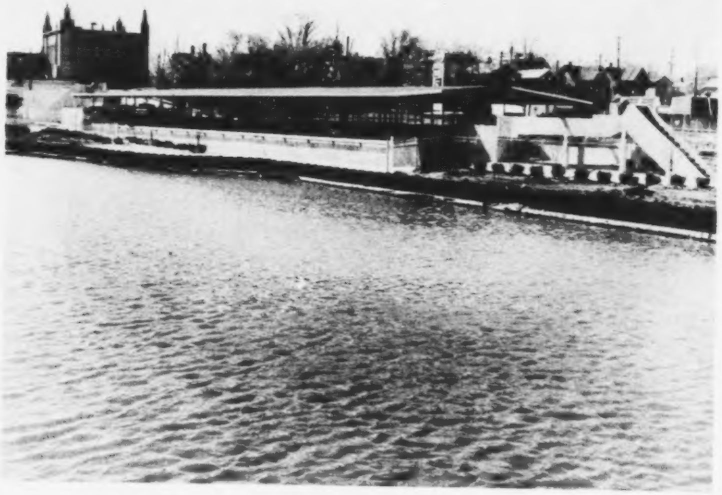


Figure 16. Site II and IIA treatment facilities.

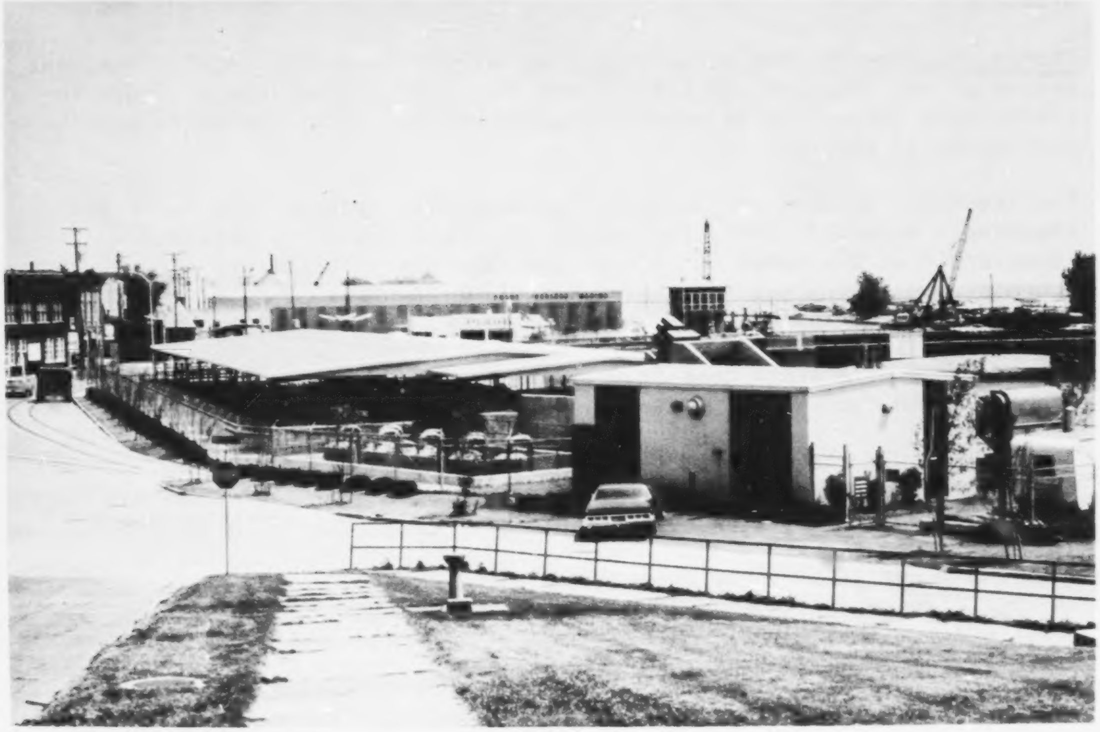


Figure 16 (continued).

a major factor in the selection of these sites. A disadvantage of these sites is the possible encroachment of Lake Michigan because of the proximity of the sites to the Lake. Studies performed in 1970, showed that, at that time, the encroachment of the Lake extended upstream to the State Street bridge area, approximately 609 m (2,000 ft) upstream of the site. This encroachment makes it difficult to determine the change in the water quality of the river brought about by operation of the treatment systems. (Further discussion of this problem can be found in Section V, ROOT RIVER MONITORING STUDIES).

Process Equipment - The screening/dissolved-air flotation (sdaf) treatment processes are identical at Sites I and II. Site IIA employs a single rotating drum screen for treatment of storm water. Site schematics are shown in Figures 17 and 18.

The treatment systems are designed for automatic startup, operation and shutdown. Automatic operation insures that the system is deployed immediately at the onset of an overflow regardless of the presence of an operator. A flow sheet indicating the major supervisory control functions is presented in Figure 19.

The presence of a storm generated discharge is detected by a bubble tube located in the screw pump wetwell. This signal initiates the sequence of events shown in Figure 19. Shutdown of the system is initiated by a low level signal in the wetwell. A cleanup cycle begins after system shutdown to ensure proper operation during the next discharge. This cycle includes backwashing of the screens and a final skimming of the flotation tansk.

At Sites I and II combined sewer overflow enters the wetwell and passes through a mechanically cleaned bar screen to the spiral screw pump (Figure 20). This bar screen has 1.9 cm (0.75 in.) openings between bars and will remove only large size debris from the flow. After each storm occurrence this debris is removed and hauled to a sanitary landfill for disposal. The screw pump discharges into a channel leading to a Parshall flume (Figures 21 and 22). Flow measurement in the flume provides a measurement of plant flow and is used to provide flow-proportional chemical feed. After flowing through the Parshall flume, the combined sewage enters the drum screens (Figure 22-25).

At all three sites, 297 micron opening (50 mesh) screens are employed to remove suspended matter in the flow. This removal is accomplished by mechaically sieving or straining the overflow or storm water as it flows through the screen. Each screen is mounted on a 2.4 m (8 ft) diameter drum. The drum is designed to be submerged to a depth of approximately 1.8 m (6 ft) and to rotate slowly so that both straining and screen backwashing can take place at the same time. When the headloss or differential through the screen becomes excessive, backwash water is drawn from the screen chamber by a backwash water pump, pumped through a wet cyclone to remove grit, and sprayed on the outer surface of the screen so as to flush solids from the inner surface (Figure 26). These solids along with the backwash water are collected in a trough and flow by gravity to the screw conveyor which delivers them to the sludge holding tanks (Figure 27). A bypass weir was

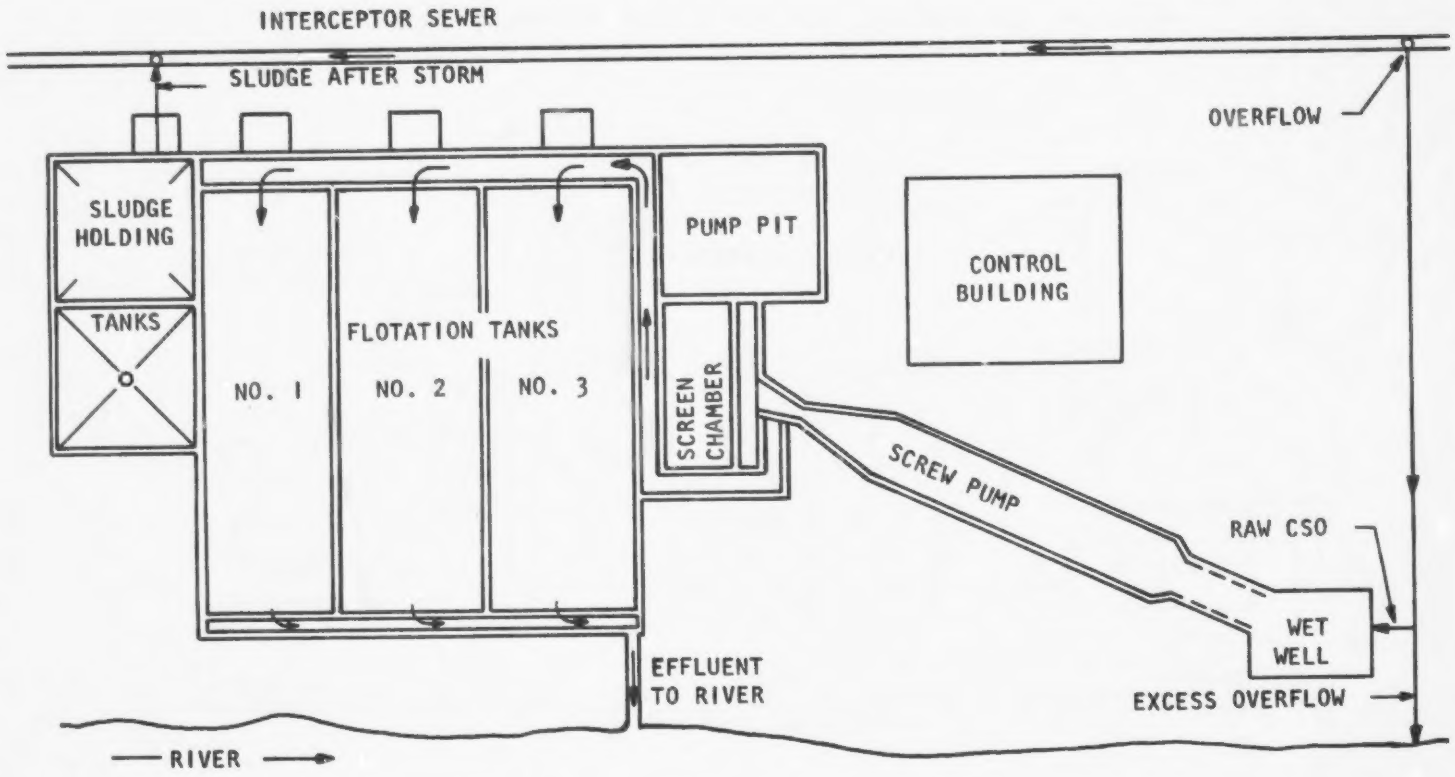


Figure 17. Schematic of Site 1 treatment facility.

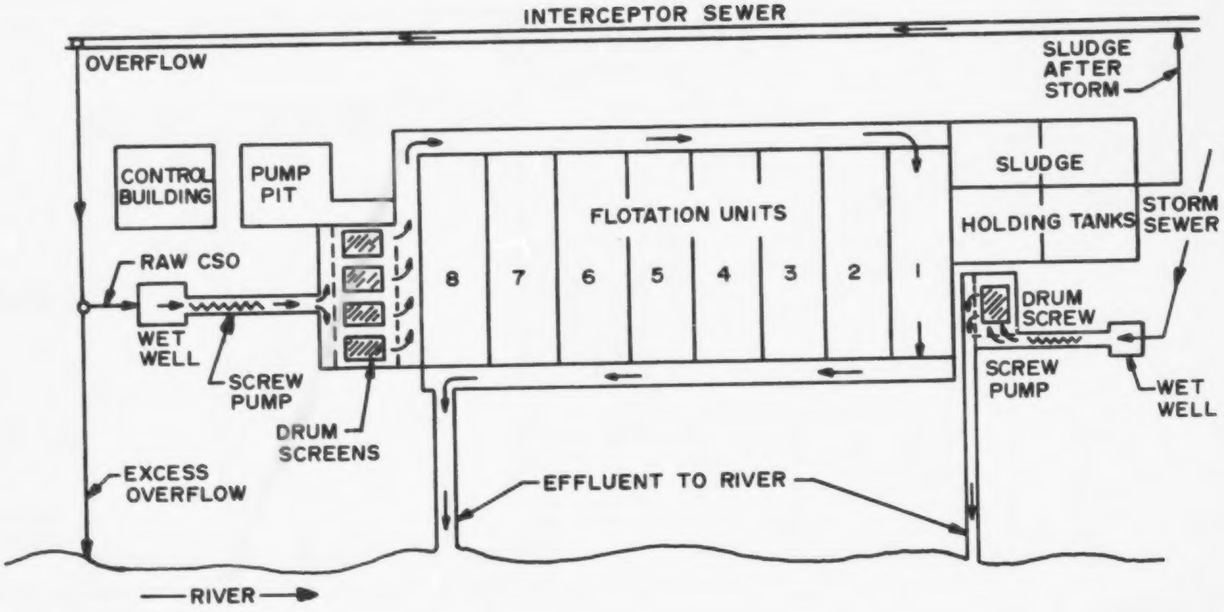
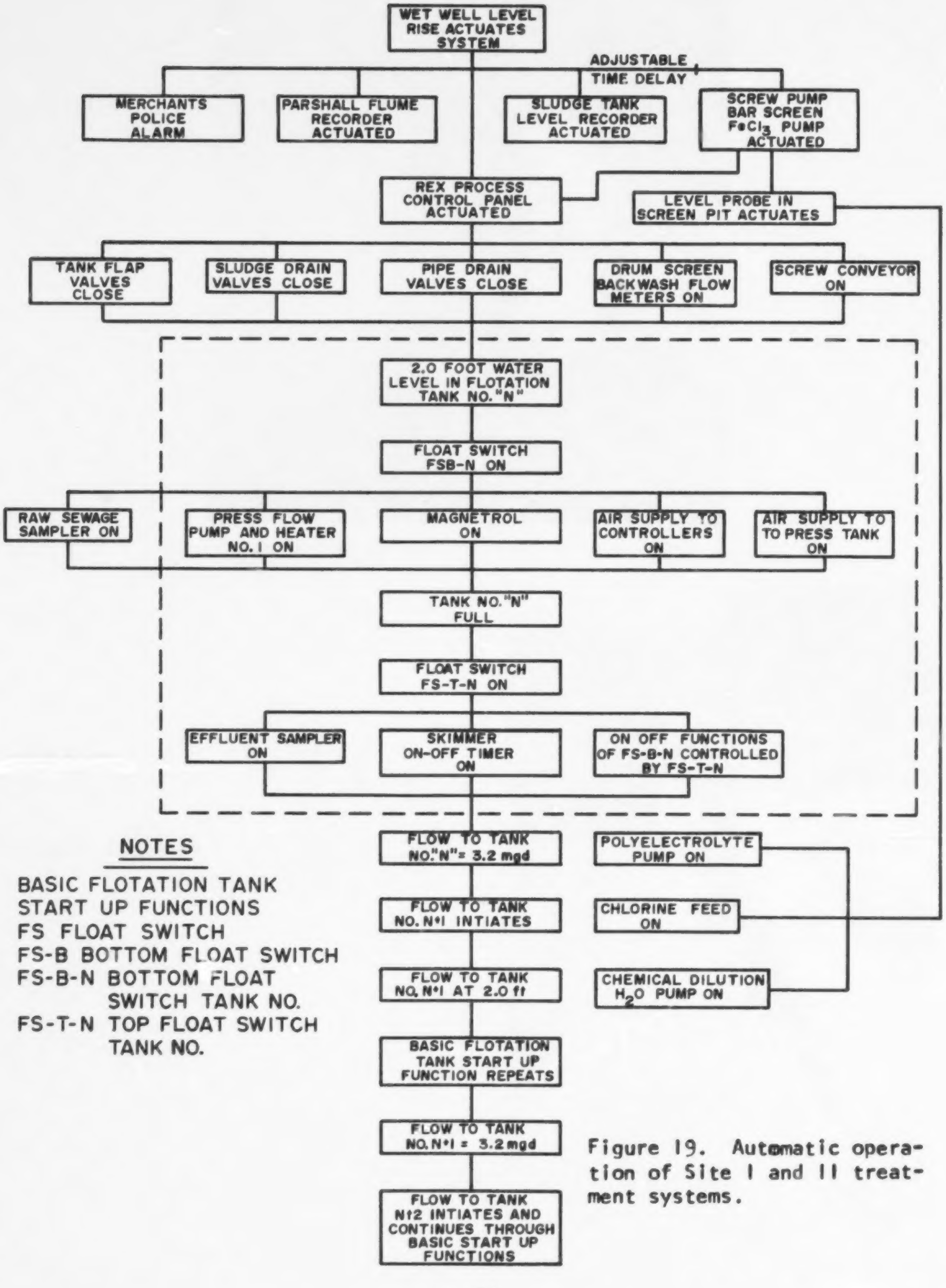


Figure 18. Schematic of Sites II and IIA treatment facilities.


WET WELL LEVEL RISE ACTUATES SYSTEM
ADJUSTABLE TIME DELAY
MERCHANTS POLICE ALARM
PARSHALL FLUME RECORDER ACTUATED
SLUDGE TANK LEVEL RECORDER ACTUATED
SCREW PUMP BAR SCREEN FeCl3 PUMP ACTUATED
REX PROCESS CONTROL PANEL ACTUATED
LEVEL PROBE IN SCREEN PIT ACTUATES
TANK FLAP VALVES CLOSE
SLUDGE DRAIN VALVES CLOSE
PIPE DRAIN VALVES CLOSE
DRUM SCREEN BACKWASH FLOW METERS ON
SCREW CONVEYOR ON
2.0 FOOT WATER LEVEL IN FLOTATION TANK NO. "N"
FLOAT SWITCH FSB-N ON
RAW SEWAGE SAMPLER ON
PRESS FLOW PUMP AND HEATER NO. 1 ON
MAGNETROL ON
AIR SUPPLY TO CONTROLLERS ON
AIR SUPPLY TO TO PRESS TANK ON
TANK NO. "N" FULL
FLOAT SWITCH FS-T-N ON
EFFLUENT SAMPLER ON
SKIMMER ON-OFF TIMER ON
ON OFF FUNCTIONS OF FS-B-N CONTROLLED BY FS-T-N
FLOW TO TANK NO. "N" = 3.2 mgd
POLYELECTROLYTE PUMP ON
FLOW TO TANK NO. N+1 INTIATES
CHLORINE FEED ON
FLOW TO TANK NO. N+1 AT 2.0 ft
CHEMICAL DILUTION H2O PUMP ON
BASIC FLOTATION TANK START UP FUNCTION REPEATS
FLOW TO TANK NO. N+1 = 3.2 mgd
FLOW TO TANK N+2 INTIATES AND CONTINUES THROUGH BASIC START UP FUNCTIONS

NOTES

BASIC FLOTATION TANK START UP FUNCTIONS
FS FLOAT SWITCH
FS-B BOTTOM FLOAT SWITCH
FS-B-N BOTTOM FLOAT SWITCH TANK NO.
FS-T-N TOP FLOAT SWITCH TANK NO.

Figure 19. Automatic operation of Site I and II treatment systems.

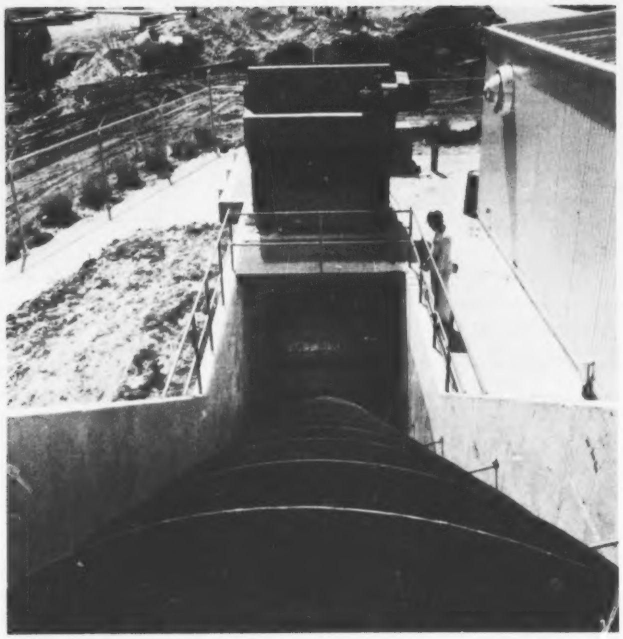


Figure 20. Site II spiral screw pump and bar screen.

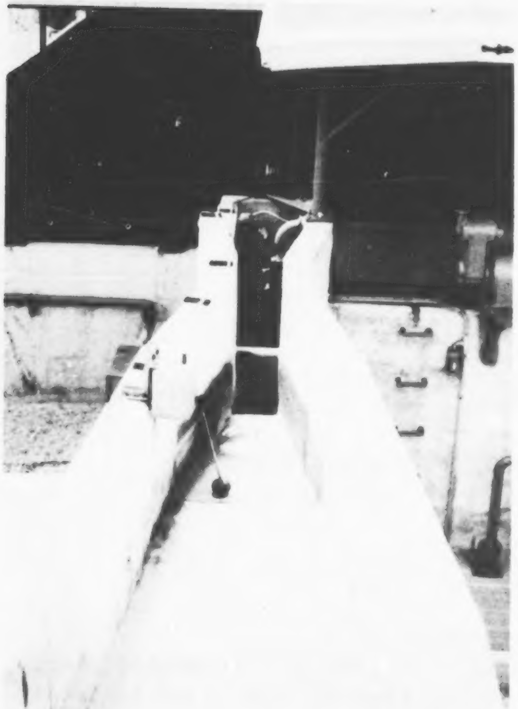


Figure 21. Site I Parshall flume and transmitter.

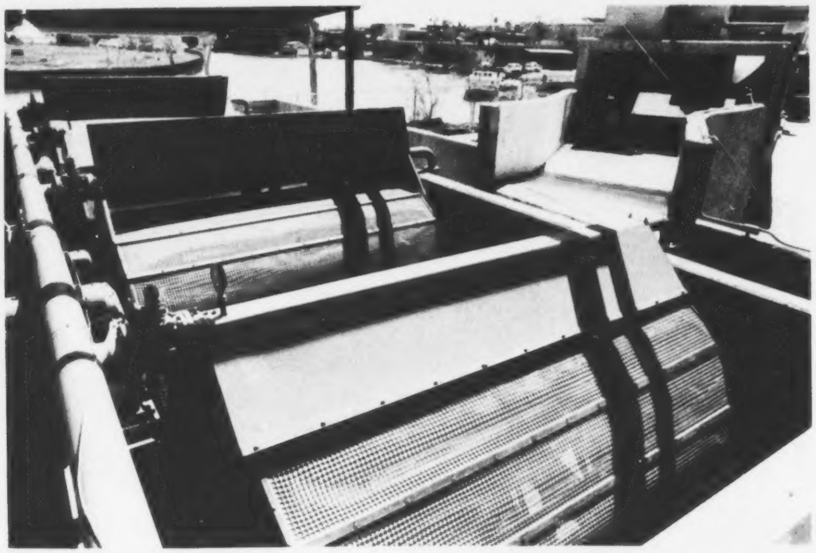


Figure 22. Site II drum screens.

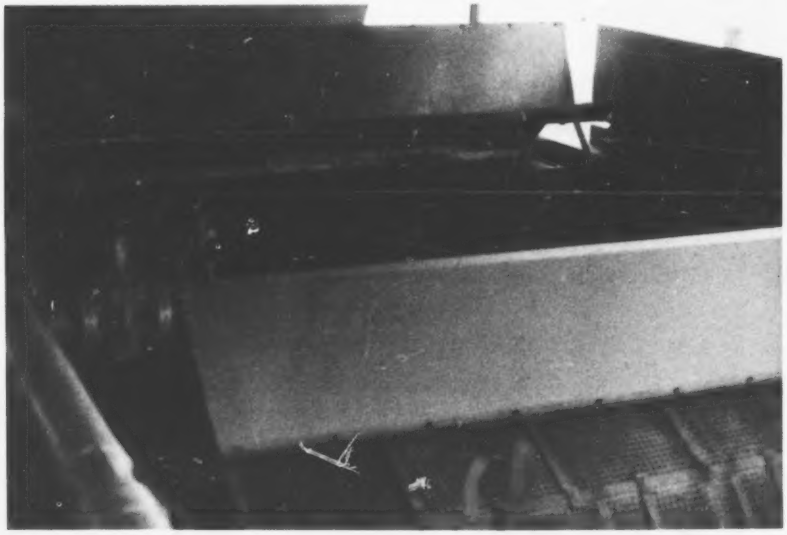


Figure 23. Site II drum screens with hold-down bars.

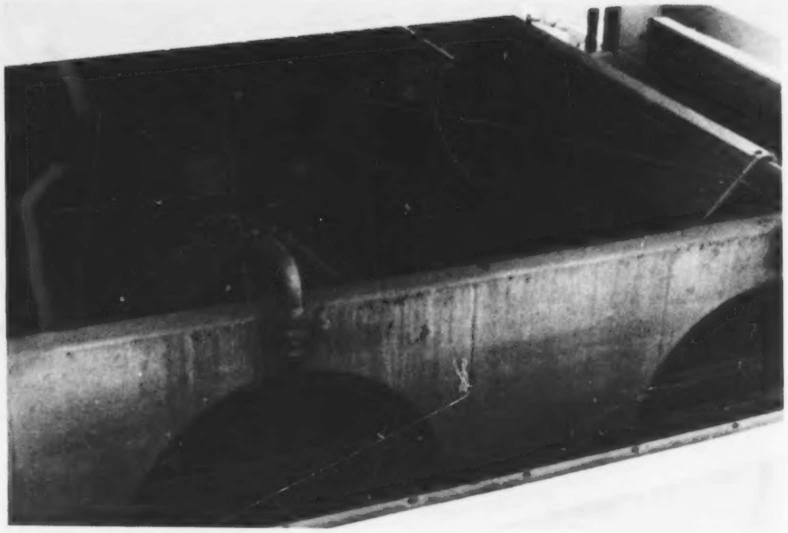


Figure 24. Site II drum screen influent channel.

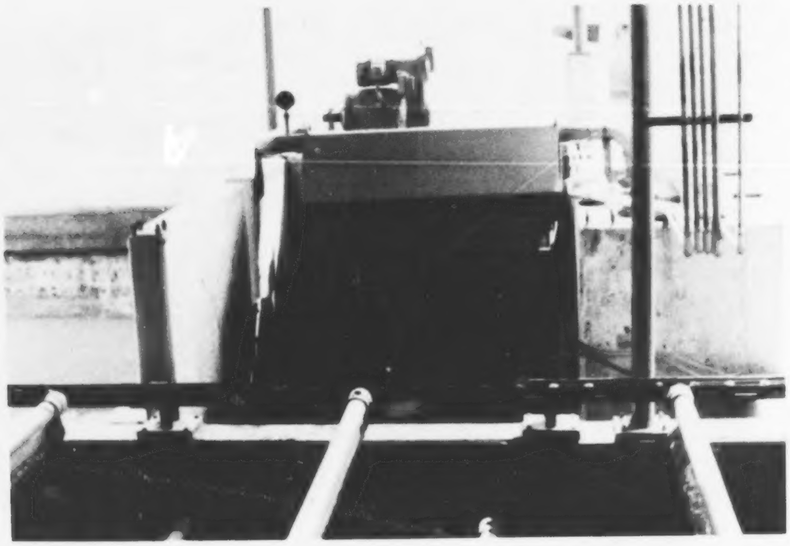


Figure 25. Site IIA drum screen.

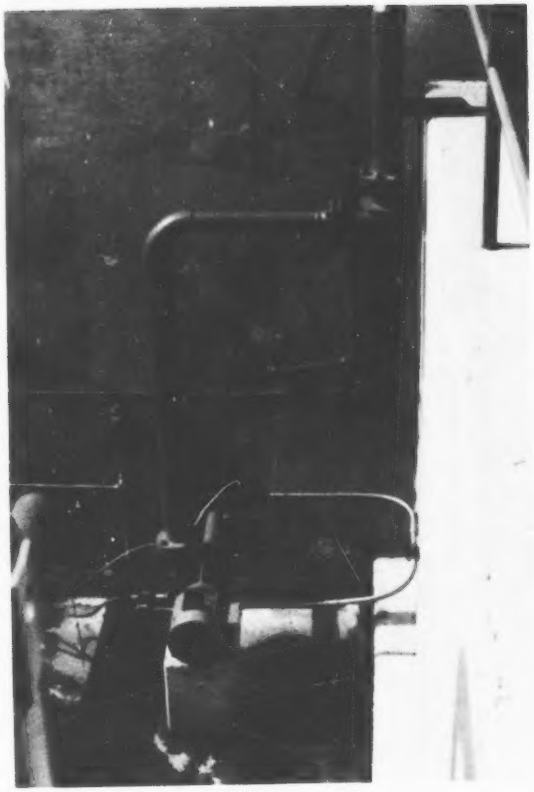


Figure 26. Site I screen backwash pump and wet cyclone.

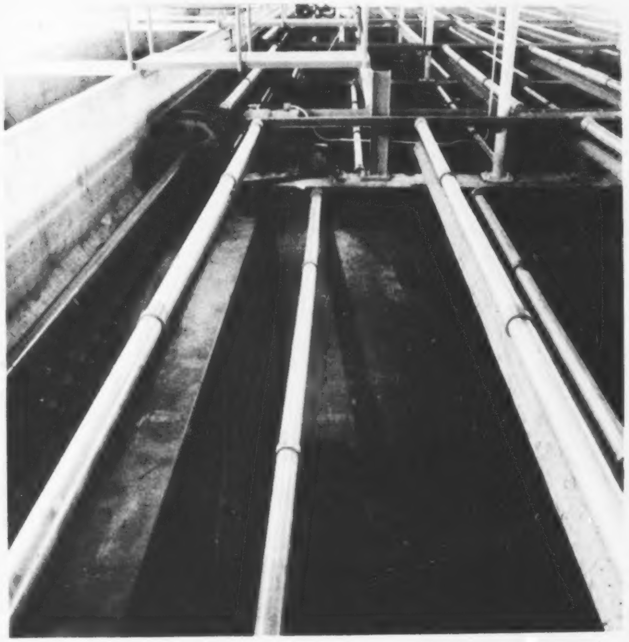


Figure 27. Site II screw conveyor and flotation tanks.

provided in the drum screen influent channel to bypass unscreened, storm-generated discharge directly into the flotation system or effluent channel (Site IIA) if all screens became clogged.

Dissolved-air flotation is the second process used to remove suspended matter from the overflow. Flotation of the solids is accomplished by introducing millions of microscopic air bubbles into the wastewater at the bottom of the tank. As these bubbles rise, they attach themselves to particles in suspension and carry them to the surface. Ferric chloride and polymer are added to the wastewater to facilitate the coagulation of particulate matter. Ferric chloride is added in the wetwell ahead of the screw pump. Polymer is added in the drum screen effluent channel. Chlorine is also added at this point for disinfection purposes.

A minimum of 20 percent of the design flow capacity is pressurized in the pressurization tank (Figures 28 and 29). Operating pressure for the tank is maintained at approximately 2.8 kg/sq cm g (40 psig) by the air panel controller (Figure 39) controlling the position of the downstream pressure control valve (Figure 31). For the optimum saturation of air in water, the pressurization tank should contain approximately 0.61 m (2 ft) of water during operation. This level is maintained in the tank by means of a level control (Figure 30) which is connected to a solenoid-operated bleed-off valve.

The air-saturated or pressurized flow is blended with the remainder of the raw flow in a mixing and flocculation zone at the influent end of each flotation tank (Figure 32). Air bubbles are formed as the pressurized flow is released to atmospheric pressure. The raw flow from the drum screens is distributed to the flotation tanks by a flow distribution channel (Figure 34). This distribution channel directs the flow sequentially into the flotation tanks in staged operation based on the rate of flow. For example, flow will be directed to tank No. 2 only when tank No. 1 reaches 70 percent of design overflow rate.

The floated sludge is periodically skimmed (timer controlled) from the top of each tank by a flight of scrapers (Figure 35) and is deposited in the screw conveyor which delivers it to the sludge holding tanks (Figures 27, 33, 36). These tanks are drained back to the interceptor sewer when the water level in the sewer has decreased to the point where the tank contents can be drained without causing an overflow at a point farther downstream in the interceptor. After the tanks are drained, they are cleansed by washing down with a firehose using river water. The washdown water pump and header system for Site II are shown in Figure 37. The intended method of operation is to drain and clean the entire system completely (sludge storage tanks and flotation tanks) after each storm. However, the system is to be deployed if a second overflow should occur before the draining and cleanup operations are completed.

Each control building is divided into two sections; one section contains the control panels, flowmeters, circuit breakers, chemical feed pumps, storage tanks, and the air compressor and dryer (Figures 38-42). The other section,

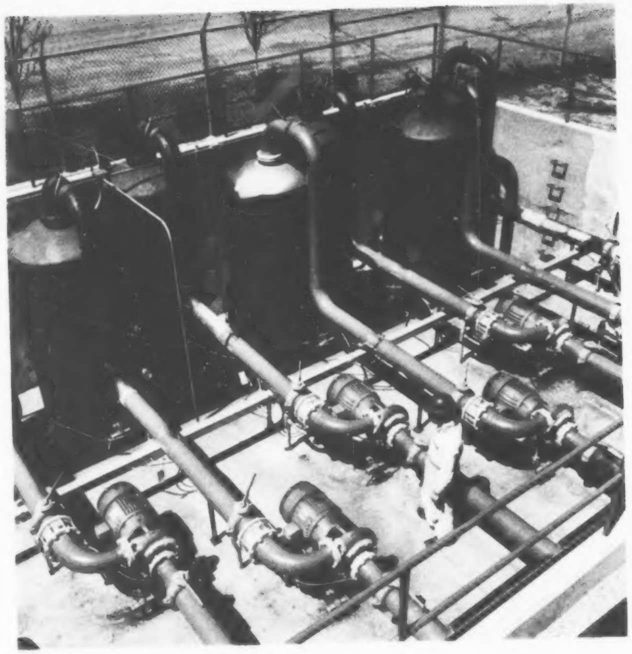


Figure 28. Site II pressurized flow system.

Figure 29. Site I pressurized flow system.

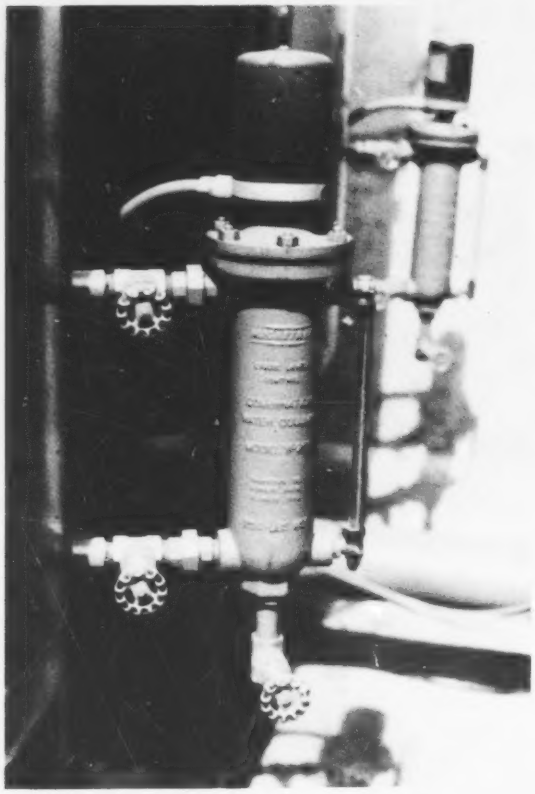


Figure 30. Pressurization tank level control and sight glass.

Figure 31. Pressure control valve in manhole ahead of flotation tank.

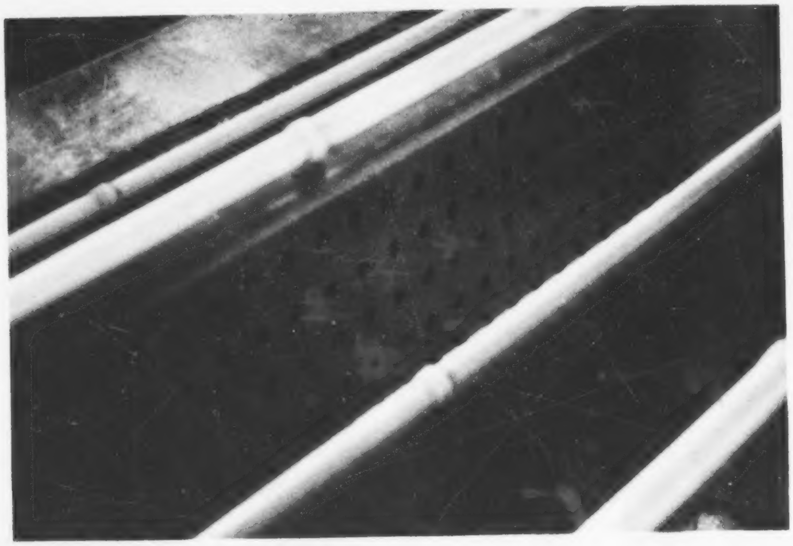


Figure 32. Mixing zone behind perforated influent baffle in flotation tank.

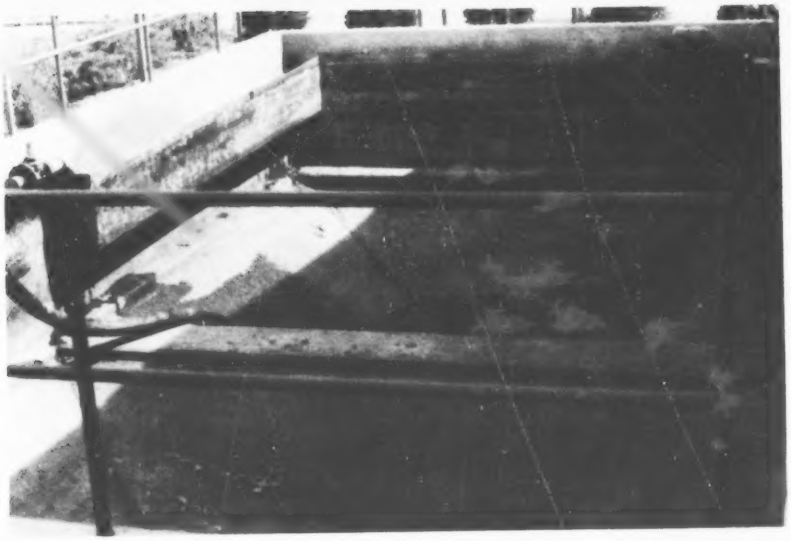


Figure 33. Site I sludge tank.

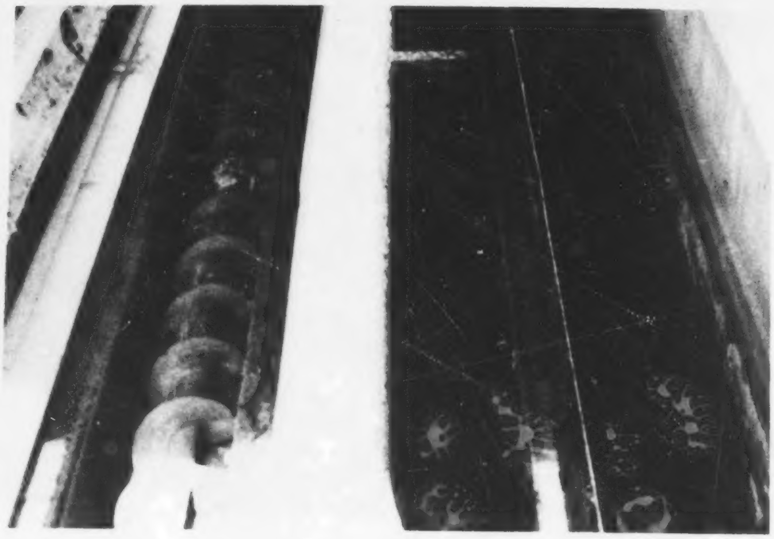


Figure 34. Site I flow distribution channel and screw conveyor.

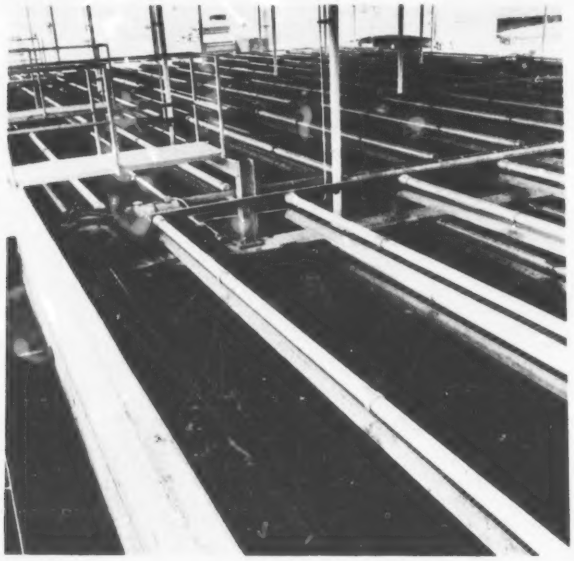


Figure 35. Site II flotation tank scraper flights.

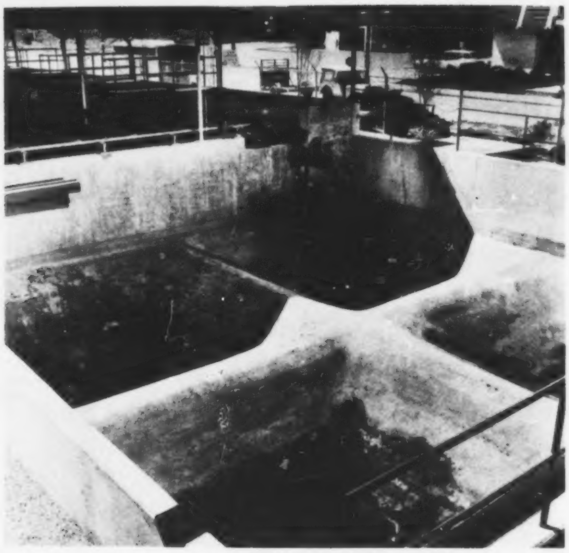


Figure 36. Site II sludge tank.

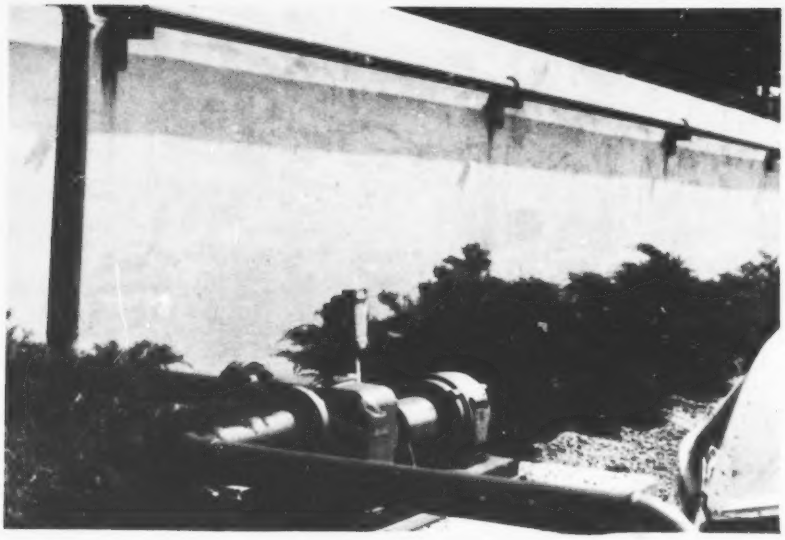


Figure 37. Site II washdown water pump.

SITE II&IIA PLANT FLOW

RATIO CONTROLLER FOR
CHEMICAL FEED

SITE II WETWELL & FLOOD
GATE LEVEL RECORDER

SITE II&IIA PLANT BYPASS

SITE IIA WETWELL &
SLUDGE TANK LEVEL
RECORDER

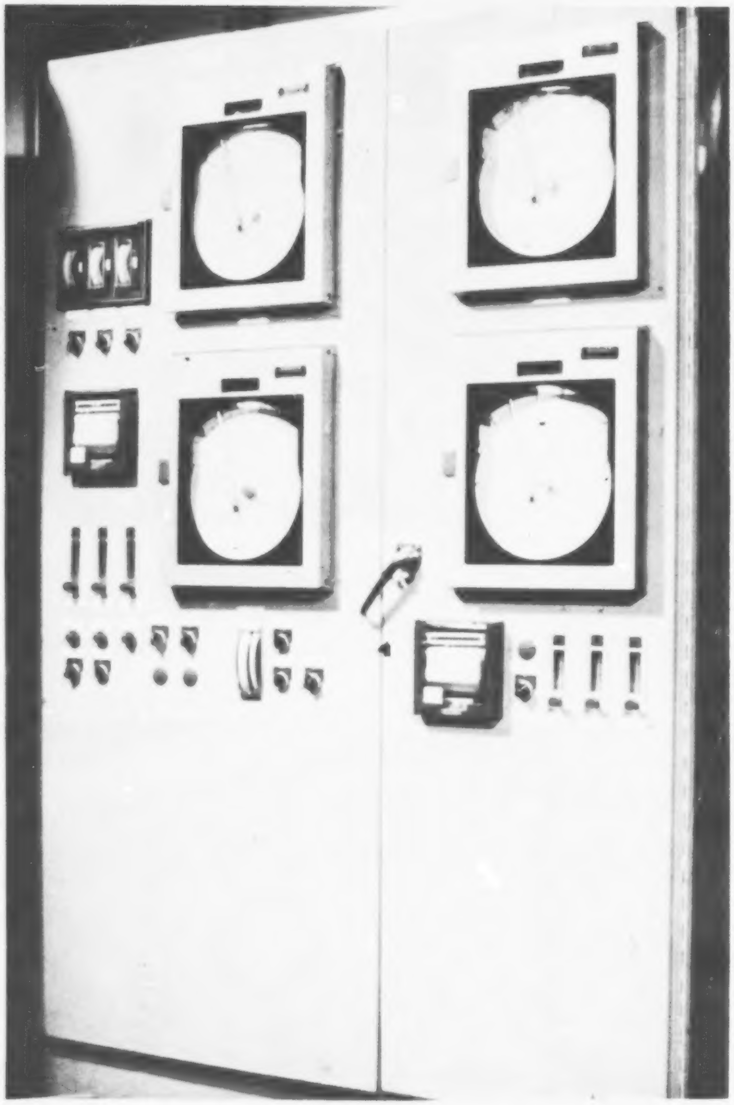


Figure 38. Site II and IIA supervisory control panel.

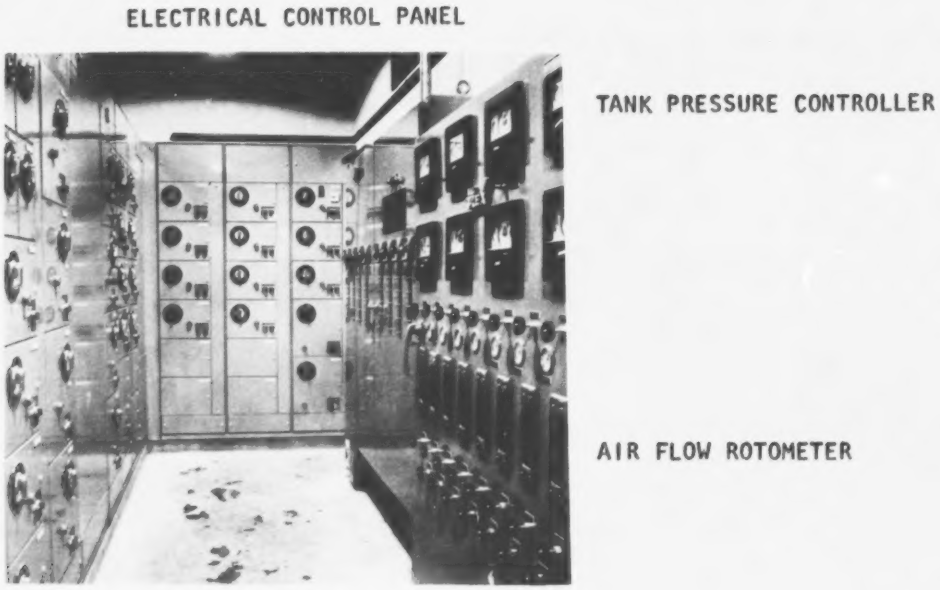


Figure 39. Site II control panel.

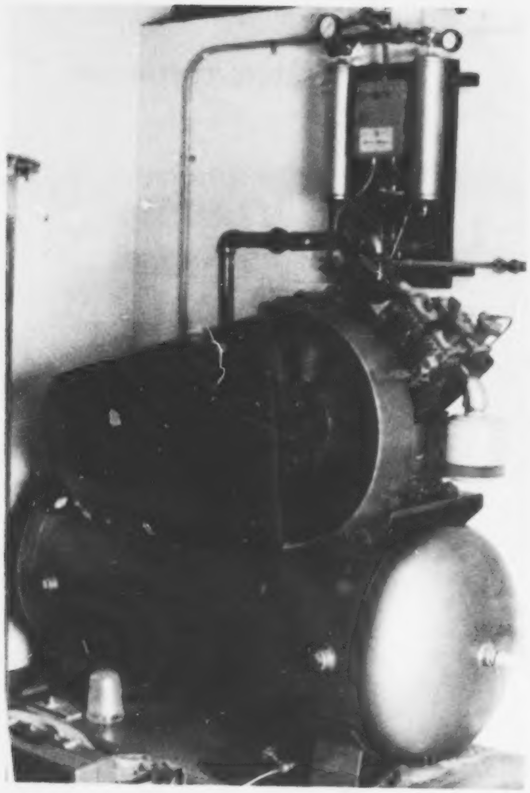


Figure 40. Site I air compressor and air dryer.

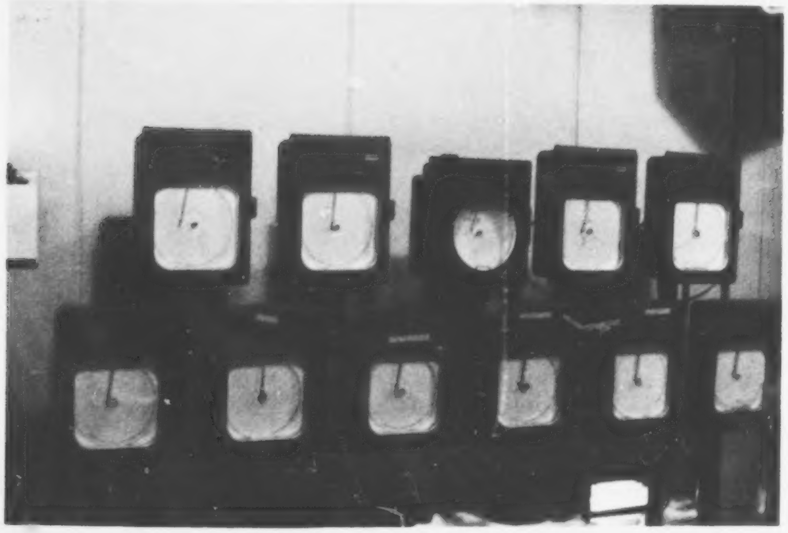
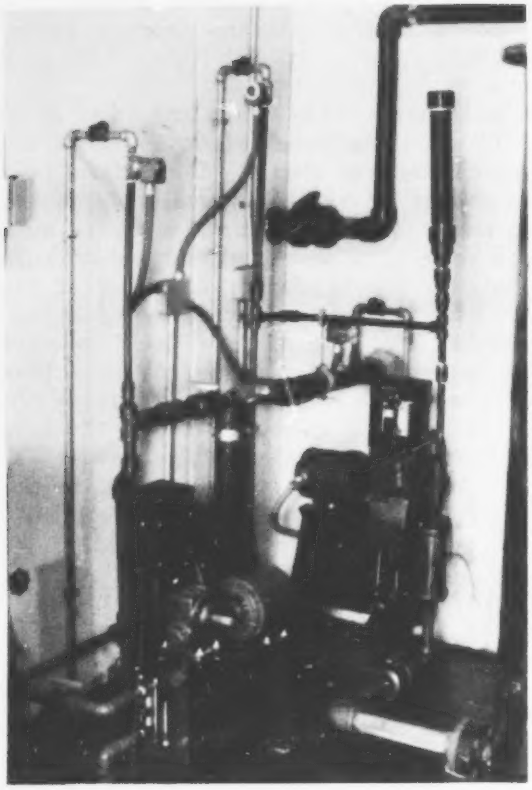


Figure 41. Site II and IIA recording flow meters (pressurized flow and as indicated).



CONTROLLER

FERRIC CHLORIDE PUMP

POLYMER PUMP

Figure 42. Site I chemical feed pumps.

separated by a cement block wall, contains the chlorine storage tanks and the chlorination equipment (Figure 43).

Flow Measurement and Control - At all three sites the plant flow is measured by means of a float in a Parshall flume. The level measurement is converted to a flow rate which is recorded on a circular chart. The volume is continuously accumulated. Flow in excess of the plant capacity is bypassed to the river at the wetwell bypass weir. This bypass is measured by means of a bubble tube. The bypass rate is also recorded on a circular chart and the total volume accumulated. The screen backwash water flow is measured by means of a Venturi. The flow rate is recorded and the total volume accumulated. For each flotation tank the pressurized flow rate is measured by means of a Venturi and recorded on a circular chart. Sludge storage tank level is measured using a bubble tube and is recorded on a strip chart recorder. This level indicates a volume which is the total of the screen backwash and floated sludge. The floated sludge volume is determined by the difference. Level measurements are also made in the wet-wells and ahead of the flood gate using bubble tubes and are recorded on strip chart recorders.

At Site 11 two additional flow control devices have been installed in the sewers. A large gate, referred to as the flood gate, was installed in the 224 cm (90 in.) trunk sewer bringing combined sewage to the Site 11 overflow point to utilize in-sewer storage in the contributing area. The position of this gate is controlled by the plant flow and by the water level upstream of the gate to utilize sewer storage when the plant is operating at capacity but at the same time to prevent any basement flooding.

The second flow control device is a sluice gate located on the 76 cm (30 in.) interceptor downstream of the overflow. This gate closes down when flotation tank No. 5 becomes full. Use of this gate is intended to minimize overflows downstream of the interceptor by maximizing the treatment rate.

Sampling Equipment - Permanent automatic samplers of a special design to facilitate the collection of storm generated discharge samples were installed at the influent and effluent end of each treatment site (Figure 10). The sampler is of the revolving arm type. Both a flexible impeller centrifugal pump and a submersible sump pump were used with the samplers. A submersible sump pump was used when the suction lift was greater than 1.8 m (6 ft) for greater reliability. The program cycle for sampling is shown in Figure 11. The time sequence can be adjusted as needed. These samplers are capable of collecting 24 discrete 1 liter samples on an adjustable time scale from about once every two minutes to once every 60 min. Where discrete samples are required, they can be obtained directly from the sampler at the specific time interval desired. For those tests for which composite sampling is desired, discrete samples can be collected at regular time intervals and composited according to flow as recorded on the Parshall flume recorder.

Design Criteria

The design criteria for the combined sewer overflow and storm water treatment systems are given in Table 11. The Environmental Sciences Division operated

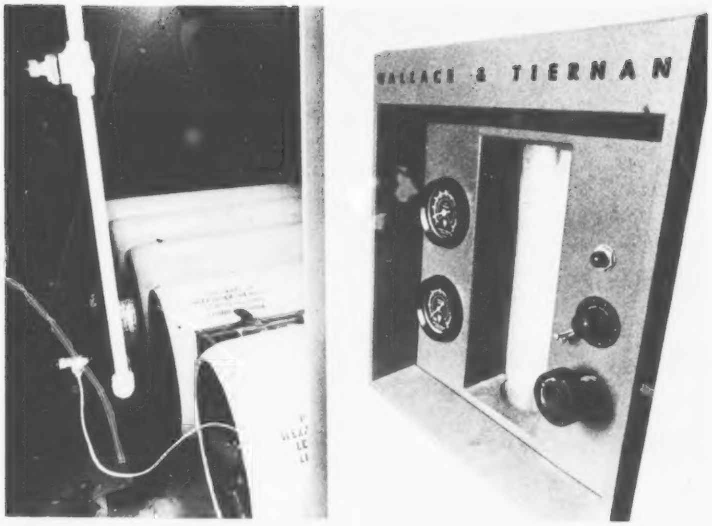


Figure 43. Site II chlorination equipment.

TABLE 11. DESIGN CRITERIA FOR COMBINED SEWER OVERFLOW AND STORM WATER TREATMENT SYSTEMS

	Site I	Site II	Site IIA
Contributing area, hectares	26.1	164.0	6.3
acres	64.5	405.2	15.5
Design storm intensity, cm/hr	1.3	1.3	1.5
in./hr	0.5	0.5	0.6
Runoff coefficient (c)	0.53	0.50	0.50
In-sewer storage, cu m	--	2,270	--
gal.	--	600,000	--
Design treatment capacity, cu m/day	53,500	168,000	14,800
mgd	14.13	44.4	3.9
Site area, sq m	1,522	3,183	(included in Site II area)
sq ft	16,384	34,263	
Bar Screens			
Channel width, m	.91	2.44	
ft	3.0	8.0	
Channel depth, m	2.44	4.57	
ft	8.0	15.0	
Flow capacity, cu m/day	53,500	168,000	
mgd	14.1	44.4	
Maximum water depth, m	1.80	2.16	
ft	5.90	7.10	
Bar size, cm	0.95	0.95	
in.	3/8	3/8	
Bar spacing (opening), cm	1.90	1.90	
in.	3/4	3/4	
Travel, m/min	2.1	2.1	
fpm	7.0	7.0	

TABLE 11. (continued)

	Site I	Site II	Site IIA
Spiral Screw Pumps			
Capacity, l/sec.	623	1,960	170
gpm	9,874	31,066	2,694
Total head, m	4.82	6.02	4.78
ft	15.81	19.75	15.68
Motor, kw	45	186	15
hp	60.3	249.30	20
Inlet fill depth, m	1.02	1.51	0.60
ft	3.34	4.97	1.97
Angle of repose	38°	30°	38°
Minimum diameter torque tube, cm	76	107	46
in.	30	42	18
Minimum flight thickness, cm	0.64	0.64	0.48
in.	1/4	1/4	3/16
Minimum spiral diameter, cm	183	244	107
in.	72	96	42
Parshall Flumes			
Throat width, cm	61	183	30
in.	24	72	12
Flow capacity, cu m/day	53,400	168,000	14,800
mgd	14.1	44.4	3.9
Drum Screens			
Number	2	4	1
Length, m	2.1	3.0	1.5
ft	7.0	9.84	4.92
Diameter, m	2.4	2.4	2.4
ft	7.87	7.87	7.87

TABLE 11. (continued)

	Site I	Site II	Site IIA
Screen mesh	50	50	50
Opening size, microns	297	297	297
Screen backwash flow, l/sec.	13.2	42.6	4.7
gpm	209.2	675.3	74.5
Backwash pressure at nozzle, kg/sq cm gauge	2.1	2.1	2.1
psi	29.8	29.8	29.8
Hydraulic loading rate, l/min/sq m	2037	2037	2037
gpm/sq ft	50	50	50
Maximum headloss capacity, cm	61	61	61
in.	24	24	24
Drum rotation speed, rpm	5	5	3
Flotation System			
No. of tanks	3	8	
Operating pressure in pressurization tank, kg/sq cm	2.8-3.5	2.8-3.5	
psi	40-50	40-50	
Tank dimensions			
length, m	16.5	15.2	
ft	54.2	50.0	
width, m	5.56	6.1	
ft	18.25	20.0	
depth, m	2.4	2.6	
ft	8.0	8.5	
Surface overflow rate at maximum design flow, l/min/sq m	135.7	157.3	
gpm/sq ft	3.33	3.86	
Detention time at maximum design flow, min.	18.2	16.5	

TABLE 11. (continued)

	Site I	Site II	Site IIA
Pressurized flow, l/min/tank	2,460	2,914	
gpm/tank	650	770	
Recycle rate, percent	25	25	
Scraper travel, m/min	0.9	0.9	
ft/min	3	3	
Compressed Air System			
Air delivery capacity, cu m/min @ 4.9 kg/sq cm	1.53	2.43	
cu ft/min @ 70 psi	54.75	86.72	
Sludge Storage			
Capacity, cu m	98	309	21
cu ft	3,500	11,030	750
(1.5% of design flow for 3 hour duration)			
Chemicals			
Maximum dosages at plant capacity			
Ferric chloride (40% $FeCl_3$ solution, mg/l	50	80	
Polyelectrolyte, mg/l (100% Nalcolyte 607 liquid)	10	15	
Chlorine, mg/l	17.0	16.2	
Chemical dilution water pump capacity, l/sec at m TDH	5.7/28	11.0/28	
gpm at ft TDH	90/92	175/92	
Chemical storage, ferric chloride, liters	6,813	11,355	
gal.	1,800	3,000	

TABLE 11. (continued)

		Site I	Site II	Site IIA
Polyelectrolyte, liters		570	1,320	
gal.		151	349	
Washdown System				
Washdown pump capacity	l/sec at kg/sq cm	6.9/3.5	6.9/3.5	
	gpm at psi	110/50	110/50	

a 19,000 cu m/day (5 mgd) screening/dissolved-air flotation pilot plant for treatment of combined sewer overflows for approximately two years under EPA Contract 14-12-40 (10). The design criteria for the Racine systems are based on the experience gained in the operation of this pilot installation.

A diagram showing the design concept of the screening/dissolved-air flotation treatment system is given in Figure 44. The various supporting systems of the Site I and II treatment plants can be divided as follows:

Pumping System

- Mechanical bar screen
- Spiral screw pump

Screening System

- Drum screens
- Backwashing system

Flotation System

- Pressurized flow pumps
- Flotation tanks
- Floated sludge removal

Chemical Addition System

- Chemical storage
- Metering pumps
- Disinfection

Instrumentation

- Measurement
- Control

Flow Control Devices

- Flood gate
- Sluice gate

Washdown System

Pumping System - At each site a spiral screw pump is used to provide the necessary head for gravity flow through the treatment units. This type of pump has a number of advantages for pumping storm generated discharges. It can operate over a wide range of suction head conditions and can pump at variable flow rates with a constant speed drive. It is nonclogging and can handle a wide variety of solids and debris without difficulty. The pump is self-priming.

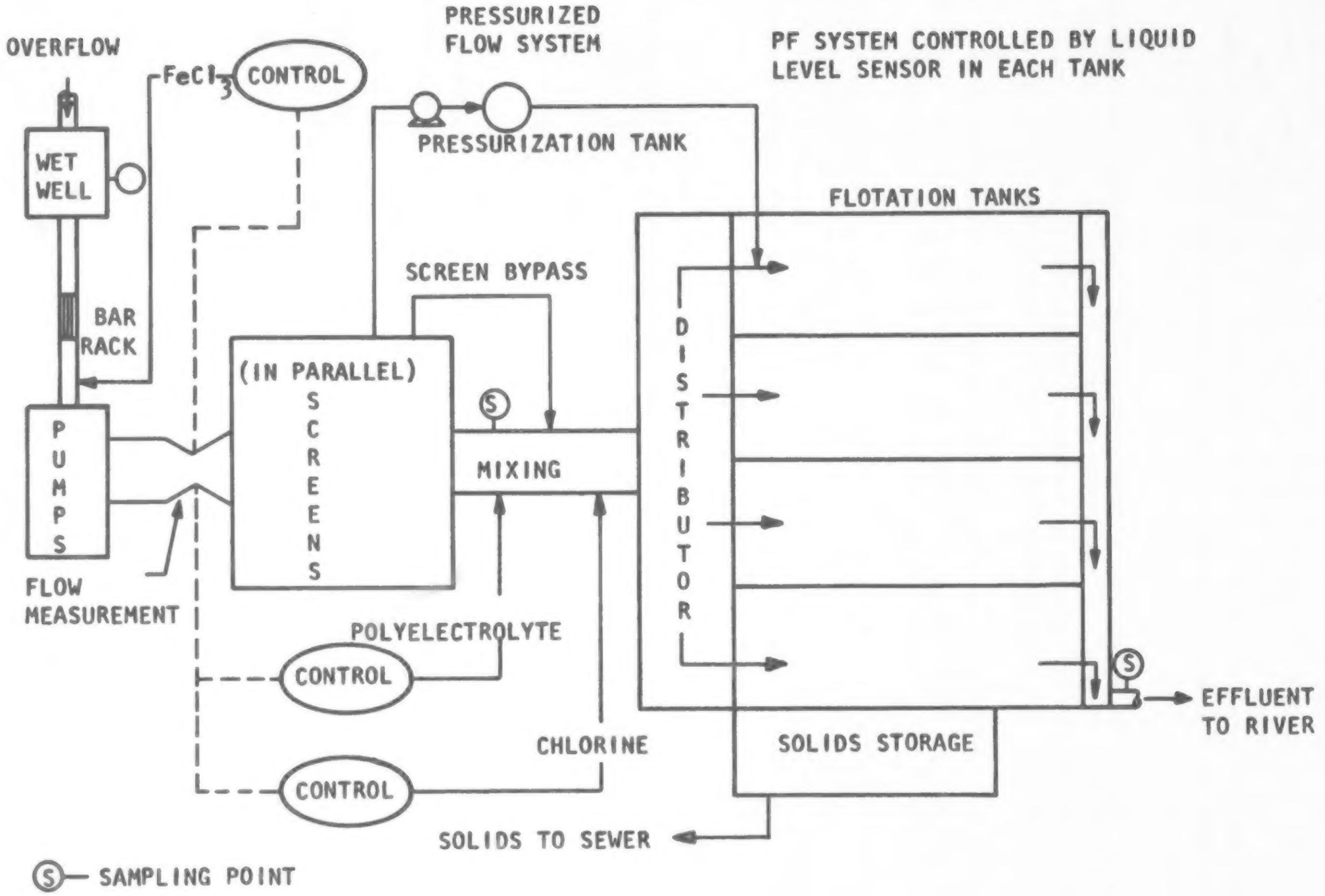


Figure 44. Conceptual design of SDAF treatment system.

A self-cleaning mechanical bar screen is utilized just upstream of the screw pumps at Sites I and II. The bar screen has 1.90 cm (0.75 in.) openings between bars and removes only large size debris from the flow which could cause problems downstream in other units of the treatment system. Debris is removed from the screen by a rake and is deposited in a hopper. The screenings are disposed of in a sanitary landfill. The type bar screen used appears to have sufficient strength to handle the solids loadings encountered.

Screening System - The screening system consists of two major components, the drum screen itself and the backwashing system. The critical design parameters associated with the drum screen include hydraulic loading, solids loading and the loss of head associated with the hydraulic loading.

Determination of the area of screen needed is based on the allowable hydraulic and solids loadings. Depending on the waste characteristics as well as on other factors, either of these variables may be limiting.

The hydraulic loading is a function of the rate of flow and area of wetted screen. The wetted area varies only slightly as the headloss through the screen changes during operation. The hydraulic loading is not affected by rotational speed of the drum. Generally, a drum screen can operate with up to 70 percent of the screen surface submerged.

Maximum design headloss or differential across the screen was 70.0 cm (27.6 in.). In practice, the drum screen was operated at a maximum headloss of from 30.5 to 40.6 cm (12 to 16 in.). The level switch controlling backwashing of the screen was set to initiate backwashing as the upper limits were approached. Drum rotation speed as well as the solids concentration in the discharge are determinants of headloss through the screen.

Drum rotation speed can be varied manually from 0 to 10 rpm. Rotation speed at Sites I and II was generally between 4.5 and 7 rpm. Rotational speed at Site IIA was between 3 and 4.5 rpm. Drum rotation is continuous and rotational speed is not varied with changing flow rates.

Solids loading is directly proportional to solids removal efficiency and inversely proportional to drum rotation speed and can be calculated using the following equation:

$$L_s = RF_s/rA_e$$

where:

L_s = solids loading, kg/100 sq m

R = screen efficiency, percent

F_s = feed solids into screen, kg/min

r = drum rotation speed, rpm

A_e = effective surface area of screen, sq m

The design solids loading was 6.8 kg of dry solids per 100 sq m (1.4 lb/100 sq ft) screen media. This design figure was based on findings at the pilot plant installation. It was found that this solids loading produced a headloss of about 33 cm (13 in.) of water.

A sketch of the drum screen configuration is shown in Figure 45. The inside of the drum is fitted with angle irons to pick up solids that do not adhere to the screen. The screen cleaning system consists of a pump, a header system and spray nozzles. Screened water is utilized to clean the screens. A hopper inside the drum collects the screenings flushed from the screen. The screenings are carried by a screw conveyor to the sludge storage tank.

The design screen backwash water rate was 3.1 l/sec/m (15 gpm/ft) of drum length. Spray nozzle pressure should be about 2.1 kg/sq cm g (30 psig). This rate was sufficient to clean a completely blinded screen. The spray nozzles utilized should provide a low mechanical pressure on the screen media but still provide good washing capability. The nozzles should have as large an opening as possible consistent with a good spray pattern. The nozzles utilized have ellipitical openings with a minor-axis dimension of 0.64 cm (0.25 in.). The nozzles are positioned about 30.5 cm (12 in.) from the drum surface. A small wet cyclone, 15.2 cm (6 in.) in diameter, is utilized to trap solids. This material drains back into the drum screen influent channel. The screening system has an automatic bypass feature. When the headloss capacity through the screen is exceeded, excess flow is bypassed around the screens to the flotation tanks.

Flotation System - The flotation system consists of the following major components: pressurized flow pumps, pressurization tanks, flotation tanks, and floated-sludge collector mechanisms.

The pressurized flow system is the heart of the dissolved-air flotation process. It includes a pump. pressurization tanks, pressure reduction valve, source of compressed air and suitable control systems. A schematic of the pressurized flow system is shown in Figure 46.

The pressurized flow system is designed to provide a rate of flow equal to 20 percent of the raw flow rate through the system at site capacity. The pressurization tank should provide maximum air-water. interface to obtain high rates of air solution. A tank without packing is recommended for treatment of combined sewer overflows and was used in this application. A packed tank would be more susceptible to plugging as a result of the solids present in the wastewater. Tanks without packing are generally fitted with an internal baffle to promote a greater air-water interface. Nominal detention time in the tank is generally about one minute. The tank should also be provided with a method to control the water level since only about 20 percent of the tank is full during operation. In this application a level control 0.61 m (2 ft) from the tank bottom activated an air bleed-off valve if the water level dropped to this point.

Design operating pressure in the system is controlled by an adjustable pressure reduction valve. The valve is positioned by use of a pneumatic

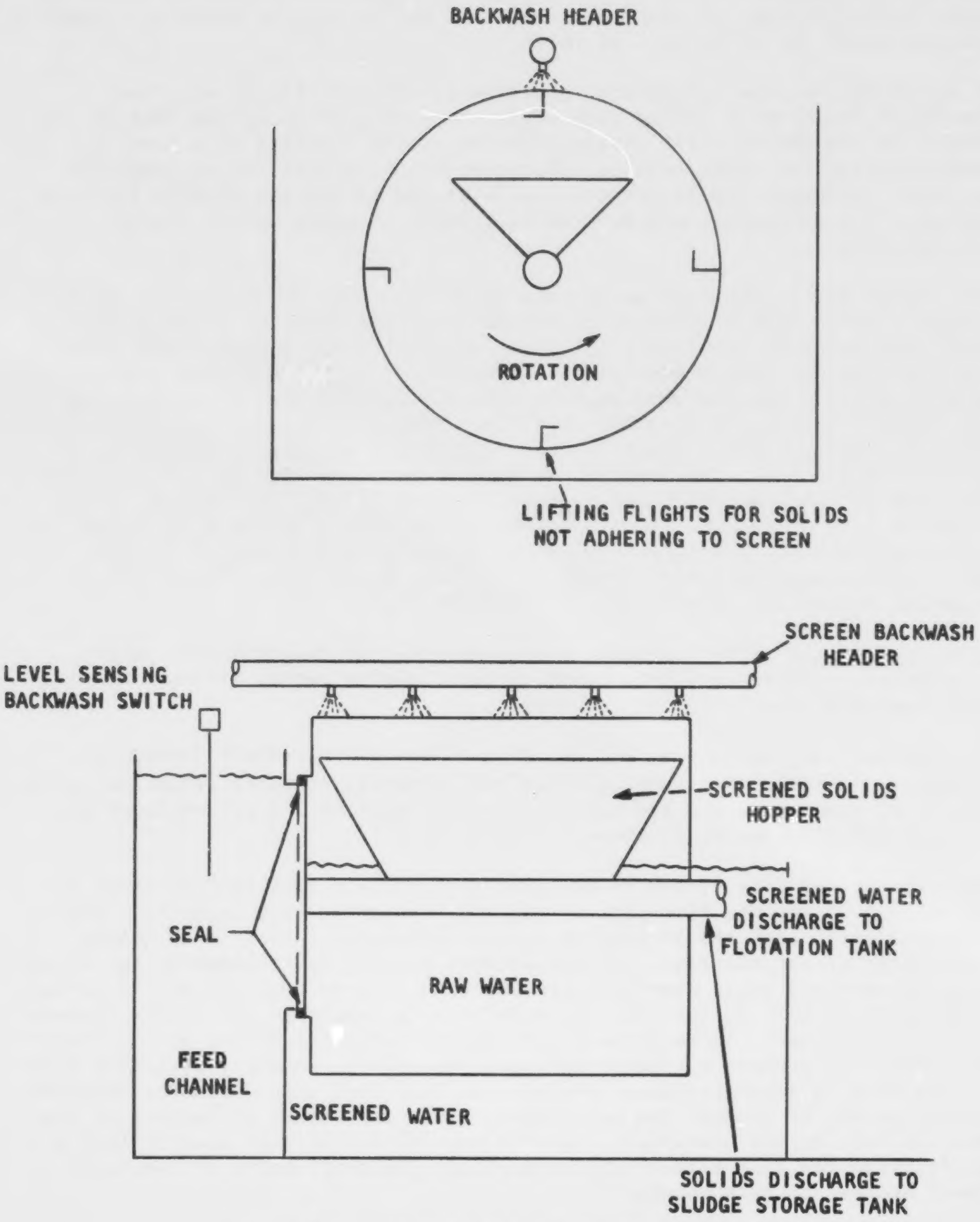

BACKWASH HEADER
ROTATION
LIFTING FLIGHTS FOR SOLIDS NOT ADHERING TO SCREEN
SCREEN BACKWASH HEADER
LEVEL SENSING BACKWASH SWITCH
SCREENED SOLIDS HOPPER
SCREENED WATER DISCHARGE TO FLOTATION TANK
SEAL
RAW WATER
FEED CHANNEL
SCREENED WATER
SOLIDS DISCHARGE TO SLUDGE STORAGE TANK

Figure 45. Sketch of rotating drum screen

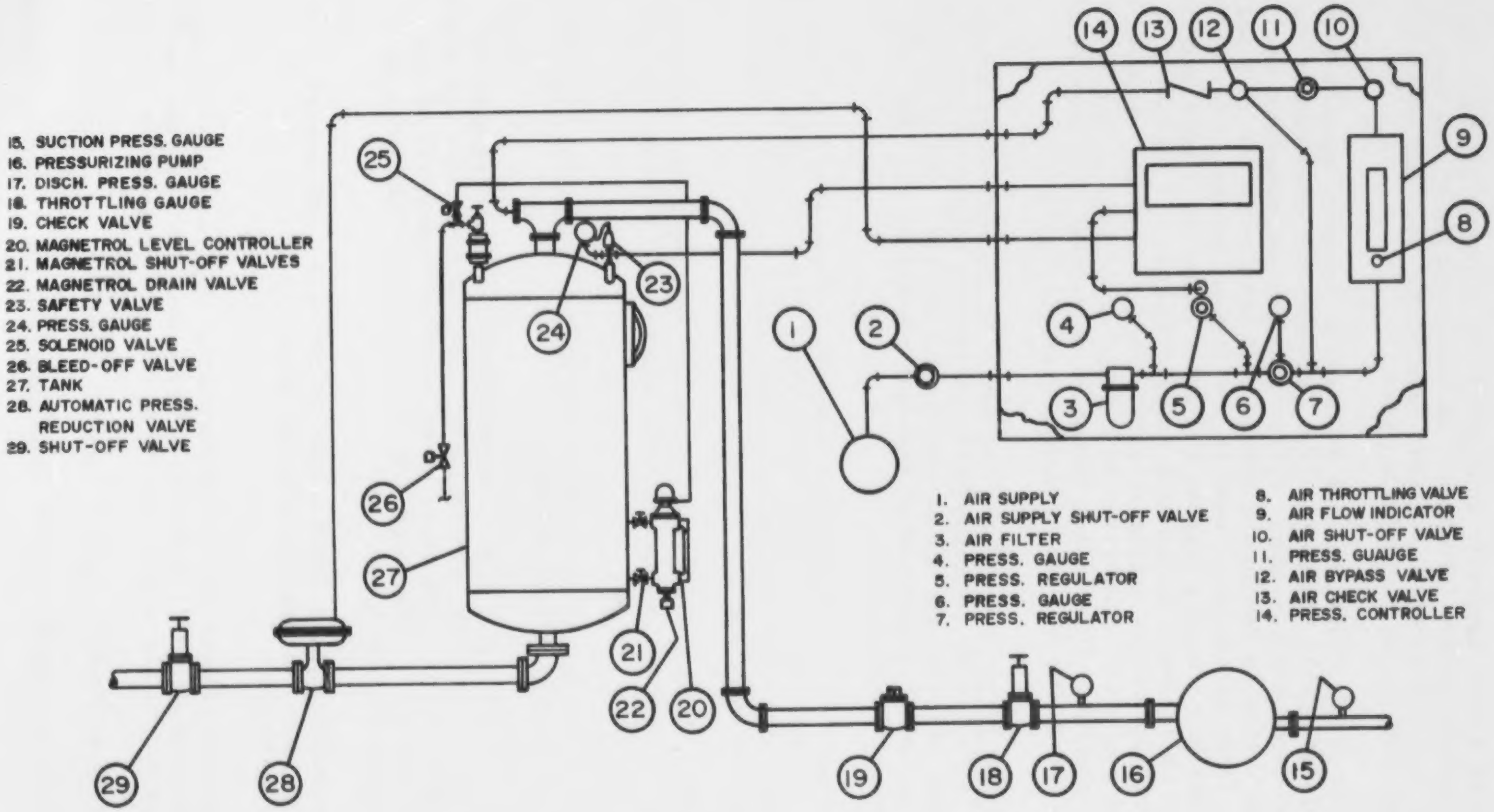


Figure 46. Schematic of pressurized flow system.

controller which allows automatic control of the system pressure. The valve provides for optimum bubble formation. An approximation of the correct air flow rate is 0.0283 standard cu m/min (1scfm) for each 378 l/min (100 gpm) of pressurized flow to the air solution tank.

The design size of the flotation tank is based principally upon American Petroleum Institute (API) standards (12). Skimmers are provided in the flotation tank to remove the scum. Bottom scrapers are sometimes utilized in a flotation system to remove any sludge that settles to the tank bottom. If 50 mesh or finer screening is used in the system, provision for bottom scrapers may not be necessary because the small amount of sludge expected can be removed while draining the tank between storms. If flotation is utilized without screening, bottom scrapers will be required. Removal of scum should be cyclically controlled by a timer or by sensing the level of the sludge blanket. This allows sludge to be removed only when required and minimizes the volume of scum requiring ultimate disposal.

Chemical Addition System - Chemicals are added to the wastewater at the points shown in Figure 44. Approximately 40 percent liquid ferric chloride ($FeCl_3$) is added ahead of the screw pump in the wetwell. The polyelectrolyte and chlorine are added downstream of the drum screens at the same point in the drum screen effluent channel.

The chemical feed rate is automatically varied with incoming flow rate by a current input/output signal between the Parshall flume transmitter and the chemical feeder. The ratio between input and output signal is adjusted to obtain the desired feed rate.

Instrumentation - There are two basic types of instrumentation utilized in the demonstration system: measurement and control. The parameters monitored are indicated in Table 12.

An explanation of the normal sequence of operation illustrates the control functions of the plant instrumentation.

Sites I and II - The Sites I and II treatment systems are designed for automatic startup, operation, and shutdown. This ensures that the system is deployed immediately at the onset of an overflow regardless of the presence of an operator. The major supervisory control functions were outlined in Figure 19 previously.

An overflow is detected by a level sensor in the screw pump wetwell. When the liquid level reaches the critical elevation, three activities are initiated:

1. A telemetry signal is sent to Merchants' Police indicating the overflow condition. Merchants' Police then notifies the appropriate people who are to report to the site and oversee operation.

2. The Envirex Process Control Panel is actuated.

3. The Envirex Process Control Panel controls the following functions:

TABLE 12. PROCESS MEASUREMENT AND CONTROL INSTRUMENTATION

Flow Measurement	Sites	Indicate/ record	Totalize	Control	Primary Instrument
Plant flow	I, II, IIA	x	x	x	Parshall Flume
Plant bypass	I, II, IIA	x	x		Weir & bubble tube
Drum screen backwash	I, II, IIA	x	x		Venturi meter
Pressurized flow	I, II	x			Venturi meter
Chlorine flow	I, II	x	x		Rotameter
Level Measurement					
Wet Well	I, II, IIA	x		x	Bubble tube (differential pressure switch-DPS)
Sludge tank	I, II	x		x	Bubble tube DPS
Drum screen - influent channel	I, II, IIA			x	Mercury DPS
Screen chamber level	I, II, IIA			x	Conductance probe/ float switch
Float tank low level	I, II, IIA			x	Float switch
Float tank high level	I, II, IIA			x	Float switch
Sewer level - flood gate		x		x	Mercury DPS
Sewer level - interceptor				x	Mercury DPS

   a. Closing the sludge drain and pipe drain valves and the flap gate in Tank No. 1.

   b. Energizing of the drum screens and backwash system.

4. The raw sewage sampler is energized and the interval timer started.

5. Following an adjustable 0 to 90 sec delay, the screw pump, bar screen, and ferric chloride feed pump are energized.

Accomplishment of these functions will put the entire treatment system up to and including the drum screens into operation.

Backwashing of the drum screens is controlled by a pressure switch in the drum screen influent channel. When the headloss through the screens increases beyond a certain critical value, the pressure switch is tripped, actuating the backwash pump. The screens are backwashed until the headloss drops, indicating the screens are clean.

Startup of the chemical feeders is initiated by an electrical conductance-type level probe located in the drum screen chamber. Critical level at this probe results in actuation of the chemical feed system:

Polyelectrolyte feed pump
Chlorinator
Chemical dilution water pump

Each of the air flotation tanks is equipped with two float switches which control their operation. These switches are identified by the following general notation:

FS-B-N and FS-T-N

where: FS-B = bottom float switch, approximately 0.61 m (2 ft) from tank bottom

FS-T = top float switch, located at effluent weir level

N = tank number in which the switch is located

Thus FS-B-2 is the bottom float switch in tank No. 2 and FS-T-2 is the top float swtich in tank No. 2.

After the combined sewage passes through the drum screen and is dosed with polyelectrolyte and chlorine, it flows through the inlet channel and begins flowing into tank No. 1. When the water level rises to the 0.61 m (2 ft) level, FS-B-1 takes control and initiates four activities:

1. Energize pressurized flow pump and meter for tank No. 1
2. Energize magnetrol
3. Supply compressed air (dry) to controllers
4. Supply compressed air (wet) to pressurization tanks

When Tank No. 1 becomes full and starts to overflow, float switch FS-T-1 actuates and three activities are initiated:

1. Energize effluent sampler
2. Energize skimmer "ON-OFF" timer
3. FS-T-1 now assumes control of the "ON-OFF" functions previously controlled by FS-B-1

Flotation Tank No. 1 is now in complete operation.

When the flow rate to Tank No. 1 reaches 70% of the design flow, which is 12,100 cu m/day (3.2 mgd), a portion of the flow is diverted to tank No. 2 through the weir splitting device in the inlet channel. The flow rate to tank No. 1 simultaneously continues to increase to the maximum design flow rate.

The startup of Tank No. 2 is initiated by FS-B-2 when the water level reaches the 0.61 m (2 ft) level. From that point on, the control is exactly the same as for tank No. 1.

Startup of all other tanks is according to the procedure outlined above, i.e., flow to the tank starts when the flow rate to the previous tank reaches 70% of design flow. The tanks continue to deploy sequentially until the design flow rate of 53,400 cu m/day (14.1 mgd) at Site I and 168,100 cu m/day (44.4 mgd) at Site II is reached. Any flow in excess of the design capacity is bypassed to the river. The rate and volume of flow which is bypassed is measured by a bubble tube and weir as described previously.

When the level sensor in the wetwell detects the end of the overflow, the following functions occur:

- Screw pump de-energized
- Bar screen rake de-energized
- Chemical feed equipment de-energized
    - Ferric chloride feeder
    - Chemical dilution water pump
    - Chlorinator
    - Polyelectrolyte feed pump
- Influent and effluent samplers de-energized

A drop in level over the effluent weir in Tank No. 1 is detected by FS-T-1 and signals the end of flow through the treatment system. This signal initiates four activities:

1. Pressurized flow pump, and air supply and controllers to tank No. 1 de-energized

2. After a suitable time delay, the Tank No. 1 skimmers are de-energized

3. The drum screens undergo a final wash to ensure a clean media and then are de-energized

4. The screw conveyor is de-energized after the drum screen wash has been completed.

The system is now shut down and ready for draining to the interceptor sewer. A differential pressure switch located in the interceptor sewer is utilized to detect when the level in the sewer has dropped to a point where the tanks can be drained without causing an overflow at a point farther downstream on the interceptor. When this preset level is reached the sludge drain valve automatically opens and the sludge storage tanks drain by gravity to the interceptor. The flap gate between flotation tank No. 1 and the sludge tanks is equipped with a delay timer so as to prevent the backflow of concentrated sludge into the flotation tanks. Thus a substantial portion of the sludge is drained prior to the draining of the flotation tanks.

It is intended practice to completely drain and clean the entire system (sludge storage tanks and flotation tanks) after each run. However, the system can and will be deployed if a second overflow should occur before the draining and cleanup operations are completed.

The differential pressure switch located in the interceptor is designed to control the sludge drain valve. If an increase in flow in the interceptor sewer occurs during the tank draining operation, the drain valve will close to prevent downstream overflows.

Site IIA - At Site IIA a simple screen is used for treatment of storm water. As with Sites I and II, this installation is designed for completely automatic startup and operation. A level sensor in the wetwell is used to initiate the following functions:

Activate Merchants' Police Alarm
Energize screw pump
Energize automatic influent sampler
Energize rotating drum screen
Energize screen backwash flowmeter.

As at Sites I and II, backwashing of the drum screen is controlled by a level probe in the drum screen influent channel. The backwash water is conveyed into the sludge holding tank. The effluent sampler is energized when the water level in the drum screen chamber trips a float switch. Shutdown of the system equipment is initiated by a low level signal in the wetwell. A cleanup cycle provides a final backwashing of the screen media prior to shutdown.

Flow Control Devices - Two additional flow control devices are installed at Site II to permit control of flow in the sewers. The sluice gate located in the 76 cm (30 in.) interceptor sewer is intended to provide a means of controlling the flow in that sewer. When the screw pump at Site II turns on, this gate closes to a preset level to control the volume of sewage flow downstream of the system. The use of this gate was designed to prevent the overflow of combined sewage between the gate and the river crossing on the south end of Ontario Street for all storms with an intensity of less than

1.3 cm/hr (0.5 in./hr). When the screw pump is deactivated, the sluice gate will open completely.

A flood gate was installed in the 228 cm (90 in.) combined sewer to enable the use of the sewer for storage. The flood gate operation in the 228 cm (90 in.) sewer is initially controlled by flow, as measured by the 182 cm (71.8 in.) Parshall flume. The normal nonstorm setting of the gate is 1.2 m (3.94 ft) above the sewer invert. At 151,400 cu m/day (40 mgd) the flood gate closes to a preset level of 0.6 m (2 ft) above the 228 cm (90 in.) sewer invert. At 168,100 cu m/day (44.4 mgd) or maximum plant flow the flood gate closes to a second position of 0.3 m (0.91 ft) above the sewer invert.

It is desired to maintain the level on the upstream side of the flood gate between 2.4 m (7.87 ft) and 2.7 m (8.85 ft) above the sewer invert. When the maximum flow of 168,100 cu m/day (44.4 mgd) is reached, control is transferred to the sewer level measurement-control system.

During the gate travel motion, all of the sensing devices will be locked out and not reinitiated until the gate has come to a complete stop. After the gate stops its travel motion, a resensing will be made of the level in the stilling well preceding the flood gate, and the gate will be driven 15 cm (6 in.) closed, or 15 cm (5.90 in.) open, or maintained at its status quo position if the level is between 2.4 m (7.87 ft) and 2.7 m (8.85 ft).

When the flow through the 182 cm (71.6 in.) Parshall flume decreases below 151,400 cu m/day (40 mgd), the flood gate is again controlled according to the incoming plant flow and the gate opens 15 cm (5.9 in.) regardless of its position, unless it is wide open. As flow drop below 113,600 cu m/day (30 mgd), the gate opens to 1.2 m (3.94 ft) above sewer level (fully open). The position of the gate is indicated on the Supervisory Control Panel.

Regardless of the flow rate to the treatment process or the level in the screw pump wetwell, when a 2.7 m (8.85 ft) level is sensed in front of the flood gate, the gate is automatically driven 15 cm (5.9 in.) further open.

Washdown System - After the tanks are drained following a treatment event, there may be solid material remaining in the sludge tanks and on the bottom of the flotation tanks. The floor of the sludge tanks and flotation tanks are sloped (0.69%) to drain the sewer and this makes it possible to wash out the tanks with a firehose. A washdown pump and 60.8 m (200 ft) of firehose have been provided at each site for this purpose.

In order to completely drain and clean out the sludge tank at Site I, it was necessary to install a submersible sump pump in the bottom of the sludge tank due to the surcharged condition of the sewers. To clean out the tanks, the sludge drain valve to the sewer was closed and the tank water was pumped with the sump pump into the sludge tank overflow weir discharging to the sewer.

Design and Construction Costs

One of the objectives of this project is to develop detailed cost information (capital, operating, and maintenance costs) to establish cost/benefit relationships for this method of treatment as compared to other treatment techniques and/or sewer separation. This information would also be helpful in evaluating the use of this treatment technique for other sites in Racine as well as in other cities.

The design, land, construction, and equipment costs for the demonstration systems are presented in detail in Appendix IV-B and summarized below.

	SITE I	SITE II	SITE IIA
Land	$ 25,100	$ 42,670	$ 1,180
Equipment & construction	368,785	705,129	21,224
Engineering	42,714	93,621	2,597
	$436,599	$841,420	$25,001

Unit prices for the system costs may be expressed as follows:

	SITE I	SITE II	SITE IIA
$/cu m/day of treatment capacity	8.16	5.01	1.69
($/mgd of treatment capacity)	30,885	18,962	6,396
based on design intensity of cm/hr	1.3	1.3	1.5
(of in./hr)	0.5	0.5	0.5
$/hectare of combined or storm sewer area	16,730	5,131	3,968
($/acre of combined or storm sewer area)	6,773	2,077	1,606

IV-3 OPERATION AND MAINTENANCE METHODOLOGY

Equipment Operation and Modifications

During the course of the project many of the operational parameters and procedures discussed under SYSTEM DESIGN AND CONSTRUCTION had to be changed or modified because of problems that were encountered in the operation of the equipment. In fact, during the first half of 1973, the common occurrence of equipment failures made it necessary to keep the sites shut down until personnel arrived at the sites. Since the travel time was about 45 minutes, significant volumes of discharge were bypassed before the treatment sites were manually turned on. It was not until November of 1973 that major equipment deficiencies were overcome to the extent that the treatment sites were placed in the automatic mode of operation.

Problems Encountered

The problems encountered during the two-year project may be briefly classified into three basic types. Type 1 problems were sporadic in occurrence and were mainly equipment breakdown that were remedied by repair work.

Examples of these problems were chemical feed-pump breakdowns, broken drive chains and level recorders. Type 2 problems were related to the design of the treatment units which could not be overcome and therefore, had to be worked into the operational procedures. These problems included hydraulic overload of the drum screens when the plant flow reached or exceeded 80% of capacity and the inability of the drum screen backwash system to completely remove the solids build-up on the screen panels. Type 3 problems also were inherent to the sites because of their design, however, these problems were overcome by making major modifications in the operational procedures. Typical examples of these problems were plant flow measurements in excess of plant capacity. This overload was reduced by lowering the spiral pump wetwell level. Other problems with the drum screens were alleviated by manually operating the backwash pumps and constantly checking the position of overlay panels and screens.

Effect of Problems on Operation - Many of the operational parameters were modified during the course of the project because of operating problems encountered and the desire to obtain optimum treatment results. The process operational parameters for each run are given in Appendix IV-C, Tables C1-C45.

Six parameters were assumed to remain constant throughout the duration of the project:

1. Backwash water pressure. The pressure was taken as 2.11 kg/sq cm g (30.4 psig) for all three sites.

2. Drum screen depth of submergence was taken as 1.45 m (4.75 ft) at Site I and 1.37 m (4.50 ft) at Sites II and IIA, because at this height the raw flow would begin flowing over the weir into the drum screen bypass channel.

3. The drum screen wetted surface areas were assumed as constant because they are calculated from the drum screen depths of submergence and a constant effective screening area. The constants used for Sites I and II were changed in 1974 because of the addition of the previously discussed lateral support bars over the drum screen panels. These support bars reduced the effective screening area and the wetted surface areas correspondingly decreased. The values for 1973 runs were 11.5, 31.7, and 4.0 sq m (124.0, 341.2 and 43.0 sq ft) for Sites I, II and IIA respectively. The 1974 values were 10.7, 29.5, and 4.0 sq m (115.2, 317.5 and 43.0 sq ft). Site IIA remained the same because support bars were not used at this site.

4. The average headloss through the drum screens were considered to be within the range stated in the design, 30-41 cm (12-18 in.) for Sites I and II and 30 cm (11.8 in.) for Site IIA.

5. The type of polyelectrolyte used throughout the project was Nalcolyte 607, a liquid cationic polyelectrolyte with a density of 1.2 kg/l (10.0 lb/gal.). The polyelectrolyte was obtained in 208 liter

(55 gal.) drums from the Nalco Chemical Co., Chicago, Illinois

6. Sewage-treatment-grade ferric chloride, obtained from K.A. Steel Chemicals, Inc., Lemont, Illinois, was used throughout the project. The concentrated solution was approximately 39 percent ferric chloride and had a density of 0.55 kg/l (4.17 lb/gal.).

For each run, average values of the following parameters were determined:

Rainfall characteristics
Flow rates and volumes
Drum screen rotational speed
Drum screen hydraulic and solids loadings
Number of flotation tanks in operation
Flotation tank overflow and pressurized flow rates
Flotation tank detention times
Pressurization tank air pressure
Skimmer flight speeds
Chemical dosages
Effluent chlorine residual
Electrical power used.

The volumes bypassed were originally intended to be an indication of volumes in excess of the treatment site capacities. However, the values were broken into two parts (before and after the run, during the run) because large volumes were being bypassed for two reasons other than exceeding the site capacities:

1. Sites did not start up automatically or were not set to start automatically and large volumes were bypassed before they were started manually.

2. Sewer discharge was occurring up to three days after the rainfall event. This situation mainly occurred at Site I in the Spring. It was considered impractical to keep the site operating that long because after four to five hours of continuous overflow, the overflow pollutional characteristics decreased to a low value and resulting treatment would be minimal.

The pressurization tank air pressure varied slightly from run to run and among the eleven individual tanks. The objective of the operating personnel was to hold the tank pressure in the range of 2.81 to 3.53 kg/sq cm g (40-50 psig).

The pressurized flow rate was not maintained at a preselected value. Instead the flow rate was the flow produced by the pressure reduction valve in maintaining an operating pressure of 2.81 to 3.53 kg/sq cm g (40-50 psig) and a water level visible in the pressurization tank sightglass.

The drum screen backwash water volumes used during site operation were theoretically dependent on the drum screen hydraulic and solids loadings

However, a relationship between these variables cannot be determined because the volumes are also greatly affected by how much plugging of the backwash nozzles, pump, and wet cyclone occurred, and how well the differential pressure switch functioned in controlling the backwash pump.

The timing (on and off) and the flight speed of the flotation tank skimmers were varied throughout the project in attempts to produce the best possible results. The final values chosen were 2 min on and 15 min off with a flight speed of 0.70 to 0.76 m/min (2.3 to 2.5 ft/min).

The drum screen rotational speeds were varied occasionally during the project. The speed was usually increased during a run when excessive amounts of drum screen bypass were occurring. The increased speed was used to decrease the solids loadings on the screens.

The chemical dosages varied throughout the project. When operation of the sites began in 1973, the selected dosages based on the results of combined sewer overflow project, EPA Contract 14-12-40 at Hawley Road in Milwaukee, Wisconsin were 20 mg/l ferric chloride and 4 mg/l polyelectrolyte (Nalco 607). After conducting bench scale tests on the combined sewer overflows from Racine in December 1973, these values were changed to 40 mg/l ferric chloride and 2 mg/l polyelectrolyte (Nalco 607). The results of these bench scale tests are given in Table 13. Due to poor operating results, the ferric chloride addition had already been increased to 40 mg/l in July 1973 before the bench scale tests were conducted.

The mean values for each operational parameter and the range of the values throughout the project duration are given in Tables 14, 15 and 16 for Sites I, II and IIA respectively.

From a comparison of the parameters there are some obvious operational differences between the sites. The ranges and means for the volumes of overflow treated by Sites I and II are similar even though the capacity of Site II is 116.6 cu m/min (30,800 gpm) compared to 36.5 cu m/min (9,643) gpm) at Site I. On the average Site I ran twice as long as Site II and the maximum run was 2479 min. This time compares with a maximum run of only 440 min at Site II. Therefore the total volume of overflow at Site I was nearly the same at Site II.

The volume bypassed during operation of Site II is very large despite the site capacity of 116.6 cu m/min (30,800 gpm) because of the periods during site operation when the bar screen rakes would jam resulting in buildup of material on the bar screen. This material would block much of the plant flow and cause large volumes to bypass the plant. If this problem had not occurred, the much larger capacity of Site II would probably have resulted in less average bypass volume during operation than was experience at Site I.

It is interesting to note, however, that the solids loading on the screens at Site II and IIA were greater than the solids loadings at Site I. This difference was caused by the higher average suspended solids concentrations

TABLE 13. RESULTS OF BENCH SCALE FLOCCULATION TESTS PERFORMED ON SITE II OVERFLOW

Date: December 4, 1973

Ferric chloride dosage, mg/l	Floc characteristics
5	Small, pin floc
10	Small, pin floc
25	Better than with 10
37.5	Good
50	Best

Polymer	Type	Polymer[a] dosage, mg/l	Floc characteristics
1A1	Anionic	0.25	Good
		0.50	Better, reforms well
		0.75	Slightly overdosed
		1.00	Slightly overdosed
607	Cationic	1.00	Fair
C-31	Cationic	1.00	Good, does not reform well
		5.00	Good, does not reform well
905-N	Nonionic	0.25	Good, poor reform
		0.50	Good, poor reform
		0.75	Better, reforms well
		1.00	Overdosed

[a] Using 50 mg/l ferric chloride.

TABLE 14. AVERAGE OPERATIONAL PARAMETERS FOR SITE I

General	Units	Mean	Range
Overflow treated	cu m	8,556	643-43,944
Duration of run	min	424	75- 2,479
Average flow rate	cu m/min	20.2	8.0- 40.3
Overflow bypassed	cu m	5,405	0-33,255
During run	cu m	554	0- 4,122
Before run	cu m	4,851	0-30,378
Power used	KWH	915	80- 4,400
Drum Screens			
Rotation speed	rpm	5.1	4.0-7.0
Backwash water volume	cu m	207	27- 1,124
Backwash water pressure	kg/sq cm	2.11	
Depth of submergence	m	1.45	
Wetted surface area	sq m	11.1	10.7-11.50
Hydraulic loading	cu m/min/sq m	1.81	0.70- 3.50
Solids loading	kg/1000 sq m	59.77	8.40-224.11
Head loss	cm	36	30-41
Flotation System			
Overflow rate	cu m/min/sq m	.092	.045-.172
Detention time	hr	0.50	0.23-0.91
Pressurized flow rate	cu m/min	3.21	1.92-4.35
Pressurization tank pressure	kg/sq cm	2.97	2.25-3.59
Skimmer time on	min	2	1-4
Skimmer time off	min	12	5-15
Skimmer flight speed	m/min	0.75	0.70-0.91
Chemical Addition			
Ferric chloride	mg/l	32	0-136
Polyelectrolyte (Nalco 607)	mg/l	2	0-4
Chlorine	mg/l	10	0-20
Effluent chlorine residual	mg/l	2.0	0-10.0

TABLE 15. AVERAGE OPERATIONAL PARAMETERS FOR SITE II

General	Units	Mean	Range
Overflow treated	cu m	9,572	984-43,376
Duration of run	min	212	30-440
Average flow rate	cu m/min	45.2	11.4-116.4
Overflow bypassed	cu m	20,397	0-137,774
During run	cu m	2,718	0-24,034
Before and after run	cu m	17,679	0-133,232
Power used	KWH	946	160-4,480
Drum Screens			
Rotation speed	rpm	5.1	3.9-7.0
Backwash water volume	cu m	301	25-1,640
Backwash water pressure	kg/sq cm	2.11	--
Depth of submergence	m	1.37	--
Wetted surface area	sq m	30.6	29.5-31.7
Hudraulic loading	cu m/min/sq m	1.39	0.39-3.67
Solids loading	kg/1000 sq m	66.01	8.87-245.70
Head loss	cm	36	30-41
Flotation System			
Overflow rate	cu m/min/sq m	.075	.019-.221
Detention time	hr	0.76	0.18-2.12
Pressurized flow rate	cu m/min	2.59	1.42-4.16
Pressurization tank pressure	kg/sq cm	2.76	2.60-2.88
Skimmer time on	min	3	1-4
Skimmer time off	min	12	5-20
Skimmer flight speed	m/min	0.72	0.70-0.91
Chemical Addition			
Ferric chloride	mg/l	34	0-147
Polyelectrolyte (Nalco 607)	mg/l	2	0-7
Chlorine	mg/l	6	0-20
Effluent chlorine residuals residuals	mg/l	1.0	0-5.3

TABLE 16. AVERAGE OPERATIONAL PARAMETERS FOR SITE 11A

General	Units	Mean	Range
Discharge treated	cu m	160	19-530
Duration of run	min	29	7-102
Average flow rate	cu m/min	5.5	2.7-9.7
Discharge bypassed	cu m	0	--
During run	cu m	0	--
Before and after run	cu m	0	--
Drum Screen			
Rotation speed	rpm	3.7	3.0-4.5
Backwash water volume	cu m	20	0-91
Backwash water pressure	kg/sq cm	2.11	--
Depth of submergence	m	1.37	--
Wetted surface area	sq m	4.0	--
Hydraulic loading	cu m/min/sq m	1.34	0.68-2.42
Solids loading	kg/1000 sq m	71.40	12.11-227.78
Head loss	cm	30	--

at Site II and IIA (280, 515, and 376 mg/l for Sites I, II, and IIA respectively).

It should also be noted at this time that the average flotation tank overflow rate was greater at Site I then at Site II, and the average flotation tank detention time was less at Site I then at Site II. This, again, was due to the fact that on the average, Site I ran at 54% of capacity and Site II ran at only 39% of capacity. The effect of this situation on operational treatment results will be considered later in this section.

Chlorine addition and chlorine residual in the effluent, on the average, are twice as great for Site I than for Site II because of the many runs for which the chlorination equipment at Site II was inoperable.

Both Sites I and II have minimum chemical additions of zero because at one time or another during the project, each particular chemical feed pump was out of operation because of mechanical difficulties.

Normal Maintenance Requirements

The development of the maintenance procedures follows from the many problems encountered in the operation of the equipment. The purpose of this portion of the report is not to be an operation and maintenance manual, but to establish basic maintenance routines that will be beneficial in eliminating or minimizing many of the equipment problems that plagued the system. A separate, detailed operation and maintenance manual was prepared for the demonstration systems.

Bar Screens - The drive chains for the rakes should be oiled in the Spring before the sites are put in operation. A sufficient supply of shear pins should be kept on hand because they are needed when the rakes become jammed and the pins shear during operation. Experience indicates that three to five shear pins may be used before the rakes are freed.

If the rakes cannot be freed during operation, maintenance time will be required to free them when sewer discharge ceases.

The wetwells, especially at Site 11, should be checked frequently for large pieces of debris that could possibly cause jamming of the bar screen rakes.

After the sites are run for a combined sewer overflow event, the debris removed from the flow by the bar screens must be disposed in a sanitary landfill or by another environmentally acceptable method.

Spiral Screw Pumps - The grease reservoir for each spiral screw pump should be filled in the Spring before the sites are put in operation and checked occasionally throughout the year.

Flow Monitoring Equipment - After each run all of the charts have to be changed and the totalizer readings should be noted on both the chart taken off and the new chart that is put on. A sufficient supply of all the different types of recording charts should be kept on hand.

The air lines to all of the Venturi flowmeters should be drained frequently to prevent plugging and after they are drained, the recording pen should be set at zero.

Drum Screens - The drum screens were a major maintenance item throughout the duration of the project. Sufficient quantities of repair parts should be kept on hand. These include:

Screening fabric
Screen panel overlays
Seals
Links for drive chains
Bearings for speed reducers
0.95 x 2.54 cm (3/8 x 1 in.) and
0.95 x 3.18 cm (3/8 x 1 1/4 in.) bolts
Fine stainless steel wire.

The drum screen channels should be checked frequently. If they are lifting up on the ends, a hole can be drilled through the end of the channel down through the outer rim of the drum screen frame. The end of the channel can then be bolted down in position.

Torn screening fabric and broken screen overlays should be replaced immediately to prevent large holes from developing that greatly reduce the effectiveness of the screening process.

During operation of the drum screens, the seals should be checked. If water is observed squirting out through the seal, it indicates that the seal has popped out of position. If possible, the seal should be wired back into position by drilling a small hole through the seal and the inside lip of the drum screen. Stainless steel or another type of noncorrosive wire is placed through the seal and the seal is secured to the lip of the drum screen. If the seal is torn, however, a new seal should be installed.

Broken drive chains should be repaired immediately because running without drum screen rotation puts excessive pressure on the drum screen panels. The chains should be oiled in the Spring before system operation begins.

If the drum screen drive mechanism becomes excessively noisy, the bearings for the speed reducer and the drive motor should be greased immediately. If the noise persists, the bearing for the speed reducer should be replaced. If this does not help, the bearing for the drive motor is probably worn and the motor will have to be removed for repairs. Frequent greasing of the drive motor and speed reducer bearings may prevent excessive wear from occurring.

Screen Backwash System - The backwash pump should be greased frequently. because of long operational times, especially at Site 1.

After every run, the backwash valving system should be changed so that all the backwash flow is pumped through the collection hoppers to flush out any remaining grit. After this flushing, the valves must be repositioned so they are ready for the next run.

The backwash water collection piping system should also be flushed out occasionally with a firehose to prevent it from becoming plugged with grit.

Pressurization System - The pressurized flow pumps should be kept greased throughout the year.

The solenoid valves for the pressurization tanks' pressure bleed-offs should be kept clean and the sightglasses for the tanks should be kept clean also so that the water level in the tanks may be easily observed during site operation.

If the pressure bleed-off valves are opening too soon or too late, the controlling mercury switch in the level control can be adjusted.

Flotation System - All of the skimmer chains should be oiled and the screw conveyor motor greased in the Spring before the sites are put in operation.

If grit begins to build up in the screw conveyor channel, it should be flushed out immediately with a firehose. Otherwise, damage may be done to the screw conveyor motor due to the excessive load caused by a large amount of grit.

The high level switch at the effluent end of each flotation tank should be checked frequently to make sure that it is free to move up and down and positioned vertically.

Chemical Addition System - A sufficient supply of chemicals should be kept on hand at all times.

The diffusers at the ends of the ferric chloride and polyelectrolyte feed lines should be cleaned out occasionally with a small wire or nail to prevent plugging. Plugged lines are indicated when the levels of the chemicals in the storage tanks do not drop during site operation. Therefore, tank level should be marked before and after each run to insure correct operation of the chemical feed system.

The entire chlorine injection system should be checked frequently for chlorine leaks by soaking a piece of cloth with aqueous ammonia and holding a cloth near all sources of possible leaks (i.e. connections, valves). If a white smoke appears to come off of the cloth there is a chlorine leak. Before attempting to stop the leak, one man should be stationed outside the chlorine room and the man working on the leak should put on the gas mask that is provided at the site.

Sampling System - The samplers should be checked during dry weather periods to make sure they are working correctly, and all hoses should be secured. A good supply of clean bottles should be kept on hand. All hoses and tubing should be kept clean.

Electrical and Pneumatic Controls - A supply of fuses and heaters should be kept on hand because blown fuses and heaters were the frequent electrical problems encountered in the operation of the sites.

A large supply of black 0.95 cm (0.375 in.) I.D. and 0.62 cm (0.25 in.) I.D. PVC tubing and the corresponding fittings should be kept on hand because of the frequent cracking of the air lines.

Rubber seats for the solenoid valves in the air control panel should also be kept on hand because they occasionally crack resulting in air leaks and loss of air pressure in the system.

All air leaks should be repaired as soon as possible.

Cleanup - Cleanup procedures required a majority of the maintenance time at the sites.

After a run, the sludge holding tanks and the flotation tanks should be drained and the solids remaining in the sludge holding tanks washed out using a firehose.

Cleaning deposited solids out of the flotation tanks requires that one man wash the tanks with a firehose while one or two men push the solids to the drain channel with shovels. By experience, it was found that during the tank cleaning, an extra man should be available to keep the drain channel clear or else solids will build up and prevent sufficient drainage. This problem was severe at Site 11, where the solids must flow all the way from tanks No. 8 through 1 to the sludge holding tanks and out the sludge drain valve. Complete cleaning of all eleven tanks by four men requires 2 to 3 days. The cleaning should be done as soon as possible after draining the tanks; otherwise odors will soon develop.

If flotation tank cleaning is not possible immediately, 0.6 to 0.9 m (2 to 3 ft) of water should be left in the tanks. This will be helpful in preventing odor problems from developing. It must be remembered that the solids increase in the bottom of the tanks with each succeeding run. Therefore, it may be more economical to clean out the tanks frequently when the volume of solids is small, than to wait until after many runs when the volume of solids is much larger and harder to remove.

The washdown pump should be greased during cleanup operations because it will be in constant use for 2 to 3 days.

It was also found that solids are deposited in the drum screen chamber during site operation. The only way to remove these solids is for one man to shovel them into a container and a second man to lift the container out of the screen chamber, and dump the solids in the screw conveyor channel. The screw conveyor can then carry them to the sludge holding tanks. Large solids get in the chamber because of holes in screen panels, leaks through the seals, and drum screen bypass flowing into the chamber, while fine solids pass through the screen and accumulate in the chamber.

During freezing weather, part of the cleanup process must be the draining of all pipes and pumps to prevent ice formation.

Maintenance Costs - Maintenance costs accounted for the largest portion of the total operating cost. Therefore, they will be discussed here in detail besides being included in the total cost of treatment discussion later in this section.

The cost of operating and maintaining the sites was 6.08¢/cu m treated (22.80¢/1000 gal.). This includes the cost of chemicals, utilities, operating personnel, and maintenance personnel. Of this total cost, 65 percent, or 3.94¢/cu m (14.90¢/1000 gal.), is due to maintenance. Reduction of maintenance costs could, therefore, greatly reduce the overall costs of treatment.

Table 17 presents a breakdown of the time spent on maintenance of the system components as previously discussed. These values were determined from the maintenance log for the months of May and June, 1974. During these two months the only maintenance on the spiral screw pumps was to check the grease reservoir and the only maintenance on the flotation system was to check the operation of the high level switches. Both items took less than 15 min and the time spent was considered as 0 man-hours.

TABLE 17. BREAKDOWN OF TYPICAL MAINTENANCE REQUIREMENTS FOR SDAF SYSTEM

Item	Maintenance time (man-hr)	Percent of total man-hr
Bar screens	12	2.3
Screw pumps	0	0.0
Flow monitoring	86	16.6
Drum screens	36.5	7.0
Backwash system	8	1.5
Pressurization system	13.5	2.6
Flotation system	0	0.0
Chemical addition system	50.5	9.7
Sampling equipment	44.5	8.6
Electrical and pneumatic controls	41	7.9
Cleanup	226	43.8
Totals	518	100.0

The difficulty of removing deposited solids from the bottom of the flotation tanks was responsible for cleanup being the major maintenance item. The design of the system makes it improbable that the cleanup time can be significantly reduced.

As the system is constructed, there is also no economical way to reduce the maintenance time required for the bar screens, drum screens, chemical addition systems, backwash system, pressurization system, and electrical and pneumatic controls. These items plus cleanup account for 74.8% of the time spent on maintenance and this time cannot be reduced without expensive design changes.

Time spent on the flow monitoring equipment can be reduced because extensive data collection is no longer necessary now that the study program is over.

Time spent on the sampling equipment could be greatly reduced by purchase of new samplers or a complete overhaul of the old samplers. The existing samplers are in a deteriorated condition after the two years of the project and near the end of the project a lot of time was spent keeping them operational.

It is believed that the maintenance costs for the bar screens, drum screens, electrical and pneumatic controls, and cleanup (61.0% of total time spent on maintenance) could be reduced greatly for future screening/dissolved-air flotation satellite plants by changing aspects of the design as recommended in the following discussion.

Recommended Future Design Considerations

Eleven basic changes should be considered for future installation:

1. An alternative source of water for the chemical dilution and screen backwash system.

2. More structural support for the drum screen panels.

3. A new design for the drum screen seals.

4. Complete separation of the drum screen bypass channel from the drum screen chamber.

5. A method of removing accumulated solids from the drum screen chamber.

6. Rakes that will clean the bar screen without jamming.

7. Air lines that will not deteriorate and are easily accessible.

8. Placement of flumes such that accurate flow measurements may be obtained.

9. An automated method of removing deposited solids from the bottom of the flotation tanks.

10. Sufficient provisions for site drainage.

11. Different types of air controllers for better control of pressurization tank pressures.

Process Water Source - The alternative source of water supply could be the municipal water system, water drawn from the receiving body of water, or final effluent. This water would be lower in solids concentrations than the screened storm water and would be used for the chemical dilution and screen backwash systems. The use of low solids water would eliminate the problems of chlorine injector plugging, backwash pump and wet cyclone plugging, and the plugging of the backwash spray nozzles. With a different source of process water, the wet cyclone may not be needed in the drum screen backwash system.

Structural Support for the Drum Screen Panels - The lateral support bars and bolts at the ends of the drum screen channels, or similar changes should be incorporated into future drum screen designs because the channels themselves do not provide sufficient support to the drum screen panels. It was found that the added lateral support reduced the amount of torn screening fabric and the number of broken screen panel overlays, resulting in a reduction of maintenance time spent on the drum screens.

Drum Screen Seals - A new type of seal design is necessary so that the seal will remain in position despite the water pressures experienced in the drum screens. If possible, a new seal material that would increase the life of the seal is desirable because replacement of a deteriorated seal is a major maintenance project.

Drum Screen Bypass Channel - Complete separation of the drum screen bypass channel and the drum screen chamber is important in order to reduce the amount of solids entering the drum screen chamber. This solids reduction would help prevent chlorine injector plugging, backwash pump and wet cyclone plugging, and backwash spray nozzle plugging if it is decided to use the screened water for process purposes.

Solids in Drum Screen Chamber - Because it was found that solids will accumulate in the drum screen chamber, a method should be provided for removing them. One possible method is to slope the floor of the chamber to a drain valve. When cleaning is necessary, the drain could be opened and the chamber washed out with a firehose.

Bar Screen Cleaning - Jamming of the bar screen rakes by debris coming into the wetwell was a major problem, therefore, it is felt that a heavy duty bar screen should be specified on future CSO treatment installations. At the time of design, it was not expected that this type of debris (PVC pipe, lumber, rubber hose) would be encountered in the overflow.

Air Lines - The air lines that are used should be resistant to deterioration in direct sunlight or completely protected from the sunlight. They should also be installed in such a way that they are easily accessible in case they crack or become plugged.

Plant Flow Monitoring - It is very important to have accurate plant flow measurement because many of the process operations are controlled by the electrical signal transmitted from the plant flow measurement device. The problems encountered with the Parshall fumes were due to installation of the flumes too close to the spiral screw pump discharges. The channel leading from the discharge to the drum screens should have sufficient length to provide nonturbulent, gravity flow before the flow is measured with a Parshall flumes.

Solids in the Flotation Tanks - An automatic system of cleaning the flotation tanks is needed to reduce the amount of maintenance time spent in manual removal of solids from the tanks. Two possible methods are provision of mechanical bottom scrapers or a steep floor slope to both the middle of the tank and to the drain end of the tank. With the sloping floor a piping system could be installed around the bottom edges of the tanks to, first, flush the solids to the middle and, then, to the drain. For both methods, each tank should be equipped with its own drain, then the solids would not have to be pushed from the tank to the sludge holding tank drain. As an alternative, a screw conveyor or flight of scrapers could be provided in the drain channel to prevent the deposition of solids.

Drainage - Site elevations should be set to provide sufficient slopes for drainage of the thick sludges that are collected in the flotation tanks and the sludge holding tanks. During the project, it was impossible to drain these tanks without the use of a firehose.

Air Controllers - Air controllers that have two pressure sensing devices should be specified for control of the pressurization tank pressure. One device is a proportional band which signals the pressure reduction valve to open or close when the pressure in the tank is above or below the desired pressure by a certain percentage (i.e, ± 5%). The second device is an automatic reset which will check the pressure once during a given time interval and will open or close the pressure reduction valve in order to bring the pressure in the tank to the desired level, i.e., 2.81 kg/sq cm g (40.5 psig). The air controllers used for this project had only the proportional band device. This resulted in problems because if the proportional band was set too sensitive the pressure reduction valve would rapidly open and close resulting in cycling of the pressurized flow rate and if it was set too high, sensitivity was lost and the range of pressure variation became too great. With the automatic reset, the proportional band could have been set high to prevent rapid cycling and yet guard against very rapid rises or drops in pressure during the reset time interval. The automatic reset would adjust the pressure to the desired level at the end of each preset time interval.

IV-4 TREATMENT RESULTS

The demonstration sites were operated for as many storm-generated discharges as possible from April 29, 1973 to September 30, 1974. During this period, 45 system runs were achieved, although all three sites were not run for all 45. Site I was run 45 times, Site II 33 times, and Site IIA 36 times.

There were two basic objectives of the project as conducted at the treatment sites:

1. To treat the largest possible volume of storm-generated-discharge.
2. To achieve the best treatment possible of a storm-generated-discharge utilizing the screening and screening/dissolved-air flotation processes.

The evaluation of the project will, therefore, be based on the degree to which these objectives were achieved and the final recommendations will cover changes or additions that may be made in order to reach these desired goals.

A system-run began when the wetwell level began to rise due to a storm-generated-discharge. When the level reached the preset high level switch, the switch began automatic site operation and an electrical signal was transmitted to the office of Merchants' Police Inc., in Milwaukee, Wisconsin. Merchants' Police, in turn, notified the personnel on-call that a high water level condition existed at the sites in Racine.

After this initial check, the personnel remained at the sites until the end of the discharge and the sites automatically turned off. If any problems were encountered during operation, every effort was made to correct them so that the run could continue until the storm-generated-discharges ceased.

After the site cleanup cycle was completed, the personnel changed all flow recording charts, recorded notes of any problems that occurred, and collected all the samples. When these chores were completed, the sites were again set for automatic operation in the event of another storm-generated-discharge, and the personnel returned to Milwaukee.

For every run, the data generated throughout the project included:

- Rainfall characteristics
- Operational parameters for the process equipment
- Treatment results based on the following analyses of influent, screened effluent, and final effluent samples:

    Total BOD
    Dissolved BOD
    Total organic carbon (TOC)
    Dissolved organic carbon (DOC)
    Total solids
    Suspended solids (SS)
    Suspended volatile solids (SVS)
    Total phosphorus (as P)
    Fecal coliforms
    pH
    Effluent chlorine residual

For specific runs during the project, the following analyses were performed as special tests:

Pesticide concentrations
Particle size distributions
Nitrogen series (organic N, NH_3, NO_2, and NO_3)
Chloride concentrations
Fecal streptococci concentrations

The operations parameters for the process equipment were discussed previously. The operational parameters along with the rainfall characteristics are given for each run in Appendix IV-C, Table C1-C45. The average values for the operational parameters over the project duration are presented in Tables 18, 19, and 20 for Sites I, II and IIA, respectively.

The results of the laboratory analyses for the influent, screened effluent, and final effluent samples from each site and individual run results are given in Appendix IV-D, Table D1-D45.

This portion of the report will cover the storm-generated-discharge characteristics and their relationship to the rainfall characteristics. The values obtained will be compared with other published results. The operating efficiency of the screening, screening/dissolved-air flotation, and chlorination processes will be discussed. Efficiency is discussed first in terms of volumes actually treated by the systems and secondly in terms of total volumes recorded. The latter volume includes the volume bypassed without being treated and is a better indication of the impact of treatment on the quality of the discharge to the Root River. The results of the special tests conducted during the project will also be covered.

Data Collection Methods

Flow Monitoring - At all three sites, flow in excess of the plant capacity is bypassed to the Root River at the wetwell bypass weir. The volume bypassed is measured by means of a bubble tube. The bypass rate is recorded on a circular chart and separately totalized. The plant flow rate is measured by a Parshall flume and is recorded on a circular chart and separately totalized. The drum screen backwash water flow rate is measured by a Venturi meter and is similarly recorded on a circular chart and separately totalized. Sludge storage tank levels are recorded on a strip chart recorder. This level indicates a volume which is the total of the backwash water and the floated sludge. The floated sludge volume can be determined by the difference.

As discussed previously, it was found during the course of the project that two of the flow monitoring devices were not accurate: the bypass weir and bubble tube for the Site I plant bypass, and the Parshall flume for Site II plant flow. Measurements were taken for both and from the measured data, correction equations were established:

Site I Bypass volume (gal.) =
0.266 (totalized vol., gal.) - 1690 (overflow time, min)

or

Site I Bypass volume (cu m) =
0.001 (totalized vol., gal.) - 6.397 (overflow time, min)

Site II Plant flow (gpm) =
3.815 (chart reading, gpm) - 16890

or

Site II Plant Flow (cu m/min) =
0.144 (Chart Reading, gpm) - 63.93

The Site II plant flow correction is used only when the chart recorder indicates flows greater than 22.7 cu m/min (6000 gpm) because the Parshall flume is accurate up to this point. Using the chart time scales, these corrected flow rates were converted to volumes which were then added to the accumulated volumes.

The corrections were applied to all the volumes obtained for Site I bypass and Site II plant flow throughout the duration of the project.

Occasionally, the backwash water volume totalizer at Site I malfunctioned. When this occurred, the total volume was calculated from the indicated circular chart flow rates and the corresponding times.

Sampling - Permanent automatic samplers were used at the influent and effluent end of each treatment site. The sampler is of the revolving arm type. Both a flexible impeller centrifugal pump and a submersible sump pump were used with the samplers. A submersible sump pump was required for greater sample reliability when the suction lift was greater than 1.8 m (6 ft). The samplers are capable of collecting 24 discrete one liter samples on an adjustable time scale from once every 2 minutes to once every 60 minutes. For the length of the project, the sampling interval was set at 10 minutes.

When discrete samples were desired, they were obtained directly from the sampler. When only a composite sample was required, the discrete samples were collected at the 10-minute intervals and composited according to the flow rate as recorded on the plant flow chart.

Exceptions to this procedure occurred when an automatic sampler did not operate correctly. In this case, grab samples were taken at regular time intervals, usually once every 20 minutes. These samples were then composited according to the plant flow rate.

Two methods of sampling the screened effluent (Site I and II) were used. During 1973, periodic grab samples were taken from the drum screen chamber. These samples were taken at 20-minute time intervals, if possible. After the run, the samples were composited according to the plant flow chart. In 1974, automatic samplers were installed to sample the screened effluent. These samplers were set to take a 500-ml sample once every 10 minutes. The discrete samples were then composited according to the plant flow chart. The screened effluent samplers did not start automatically, whereas the influent and effluent samplers did. The former had to be started when operating personnel arrived at the sites.

Drum screen backwash water and floated sludge samples were obtained manually. One or two grab samples of each were taken whenever possible during a run.

Given the physical layout of the systems, it was very difficult to obtain a representative sample of the floated sludge. It was found that the solids concentration was highly dependent on how and when the sample was taken. Similarly, it was very difficult to obtain a representative drum screen backwash water sample.

Two factors caused difficulty in obtaining these two samples:

1. The long periods of time when the backwash pump was on, especially at the beginning of a run. Since the only sampling location was at the discharge of the screw conveyor where both backwash water and floated sludge discharge, personnel had to be present to obtain a floated sludge sample when the backwash pump was off and the floated sludge was being skimmed off the flotation tanks. For a backwash water sample, they had to be present at the screw conveyor discharge when the backwash pump was on, although floated sludge was not being skimmed off any of the tanks. Because the backwash pump was usually on early in a run, obtaining of a floated sludge sample was extremely difficult.

2. The rapid filling of the sludge holding tanks. This problem meant that the samples had to be obtained in the first hour and a half of operation, because when the sludge tanks were full, the screw conveyor discharge was submerged. Since operational problems at the sites usually occurred at the beginning of a run, time was not available to obtain a sample at the right instant (backwash pump off and tank scrapers on, or all scrapers off and backwash pump on) before the sludge holding tanks were filled. Other contributing factors were the time required to develop a good floated sludge blanket and the fact that in many cases the sludge holding tanks were full or partially full from a previous run.

The purpose of collecting samples of floated sludge and drum screen backwash was to provide the input necessary for performing a mass balance for the treatment system. However, a mass balance could be performed by using the influent and effluent solids concentrations. It was felt that the latter method was the better one considering the lack of precision in sampling. Collection of floated sludge and drum screen backwash water samples was eliminated during 1974 operations. The values obtained during 1973 will be presented and briefly discussed later.

Sample Preservation - After each run, the samples were removed from the samplers. When only composites were required, the samples were composited according to the plant flow chart to form one composite sample and the composite sample bottle was then capped and labeled. When discrete samples were desired, a portion of each discrete sample was taken for the composite sample and then the composite and discretes were capped and labeled. In addition, a portion of each effluent sample, Site I and II, both discrete and composite, was placed in a sterilized bottle containing sodium thiosulfate to neutralize the effluent of the chlorine. These samples were used for the fecal (composite) and total (discrete) coliform analyses.

When all the samples were collected and labeled they were taken back to Milwaukee. On arrival, the samples were immediately taken to the laboratory. If it was before 11:00 pm, the sample analyses began immediately. If after 11:00 pm, the samples were refrigerated and the analyses were begun at 8:00 am the next morning. The samples were kept refrigerated at 4 C until all of the analyses were completed and the results checked by the responsible personnel.

Sample Analysis - All analyses were performed at the Milwaukee laboratory of the Environmental Sciences Division with two exceptions:

1. Temperatures and chlorine residuals in the effluent were determined by personnel at the treatment sites.
2. The special pesticide concentration tests were conducted by the U.S. EPA, Region V Laboratory in Chicago, Illinois, and Limnetics, Inc., Milwaukee, Wisconsin.

The analytical methods utilized are referenced in Appendix IV-E.

Characteristics of Storm Generated Discharges

Combined Sewer Overflows - The volumes of combined sewer overflow arriving at the sites are largely dependent on the characteristics of the rainfall event. This relationship was looked at in great detail during the project, especially from May to September, 1974. Every attempt was made to obtain accurate raingage charts, plant flow charts, and plant bypass charts for every rainfall event that caused combined sewer overflow. This meant that the rain gage charts and the plant bypass charts were collected even when the sites wre not operated.

A discussion of the raingages, the gaging network, and rainfall records for the project duration are presented in Section V, ROOT RIVER MONITORING STUDIES.

Combined sewer overflow hydrographs and rainfall hyetographs for selected storms are presented and discussed in Section VI, STORM WATER MANAGEMENT MODEL.

The runs for which all of the necessary data were available are given in Table 18. The total overflow volumes are the volumes treated by the sites plus any volumes bypassed. The rainfall characteristics are based on the Theissen values obtained from the three raingage locations. The days since the last system run, that is, the days since the last rainfall large enough to cause an overflow, are included to give an indication of the sewer capacity available before an overflow will occur.

Regression analyses (13) were run on the data in an attempt to develop relationships between the overflow volumes and the rainfall characteristics. The resulting correlation coefficients for the different relationships are given in Table 19.

TABLE 18. RAINFALL CHARACTERISTICS AND COMBINED SEWER OVERFLOW VOLUMES

Run No.	Total overflow Site I		Total overflow Site II		Total rainfall		Average intensity		Maximum intensity		Days since last run	
	cu m	gal. $\times 10^4$	cu m	gal. $\times 10^4$	cm	in.	cm/hr	in./hr	cm/hr	in./hr	Site I	Site II
24	13344	353	98577	2604	1.57	0.62	0.32	0.13	6.10	2.40	6	6
25	5972	158	5942	157	0.66	0.26	0.20	0.08	0.46	0.18	15	15
26	54769	1447	49015	1295	3.81	1.50	0.85	0.33	5.08	2.00	10	10
27	946	25	1173	31	0.30	0.12	0.20	0.08	0.33	0.13	7	7
28	3374	89	6549	173	1.09	0.43	0.10	0.04	0.18	0.07	3	3
29	19474	515	27914	737	1.14	0.45	0.15	0.06	3.05	1.20	3	3
30	3203	85	3066	81	0.51	0.20	0.61	0.24	2.29	0.90	2	2
31	24565	649	19757	522	1.32	0.52	0.33	0.13	1.22	0.48	1	1
32	21940	580	39024	1031	1.98	0.78	0.40	0.16	0.56	0.22	2	2
33	37805	999	13967	369	0.53	0.21	0.14	0.06	3.66	1.44	0	0
34	4624	122	5337	141	0.64	0.25	0.30	0.12	6.98	2.75	5	5
35	43681	1154	148940	3935	1.68	0.66	0.12	0.05	3.43	1.35	0	0
36	1892	50	--	--	0.53	0.21	0.12	0.05	2.29	0.90	16	--
37	9764	258	5905	156	1.52	0.60	0.28	0.11	3.81	1.50	0	16
38	48509	1282	145269	3838	2.59	1.02	0.52	0.20	3.81	1.50	3	3
39	4413	117	2044	54	0.41	0.16	0.41	0.16	5.72	2.25	2	2
41	5223	138	9841	260	1.22	0.48	0.13	0.05	0.41	0.16	7	7
42	1892	50	3293	87	0.79	0.31	0.79	0.31	3.25	1.28	12	12
43	908	24	1703	45	0.33	0.13	0.12	0.05	0.69	0.27	3	3
44	643	17	--	--	0.51	0.20	0.20	0.08	1.07	0.42	16	--
45	4958	131	6170	163	1.42	0.56	0.24	0.09	2.29	0.90	6	22

TABLE 19. CORRELATION COEFFICIENTS FOR REGRESSION ANALYSES PERFORMED ON OVERFLOW VOLUMES AND RAINFALL CHARACTERISTICS

Independent variables	Total overflow volume	
	Site I	Site II
Total rainfall	.770[a]	.596[a]
Total rainfall and average intensity	.775[a]	.629[b]
Total rainfall, average intensity and maximum intensity	.791[a]	.676[b]
Total rainfall, average intensity, maximum intensity, and days since last run	.814[a]	.688[b]
Total rainfall and maximum intensity	.779[a]	.621[b]
Total rainfall, maximum intensity, and days since last run	.810[a]	.649[b]
Total rainfall and days since last run	.806[a]	.633[a]

a. Significant at the 99 percent confidence level

b. Significant at the 95 percent confidence level

Correlations above the 99 percent confidence level were obtained for all relationships to volumes at Site I.

The best correlation coefficient, 0.770, was obtained for the relationship between the total overflow volume at Site I and the total rainfall. This coefficient is not the largest when numerically compared to others. However, it is the most significant when the correlation coefficients are compared to their corresponding critical values. The critical values are based on the degrees of freedom for the sample group, and the number of independent variables being considered. The degrees of freedom equal the sample size, n, minus the total number of variables, both independent and dependent. The only exception is for a one-to-one relationship, then the degrees of freedom equal n-1. Using the degrees of freedom and the number of independent variables, the critical value for the desired level of confidence can be obtained from a statistical table (14) (17).

In this process, as more independent variables are introduced, the degrees of freedom decrease and the critical values increase. Only slight increases in the correlation coefficient when new variables are introduced are actually reducing its overall significance. This is why the value of 0.770 for the relationship between rainfall amount and total overflow is the most significant.

The linear regression equation for the relationship is:

$$V_1 = 15502R - 3270$$

where V_1 = total overflow volume at Site I, cu m
R = total rainfall, cm

This equation predicts that it requires 0.20 cm (0.08 in.) of rain to cause the combined sewer to overflow at Site I, and that a volume of 3938 cu m (1,040,000 gal.) can be expected at Site I for each 0.25 cm (0.10 in.) of rain that falls after the first 0.20 cm (0.08 in.).

The application of the above criteria to the correlation coefficients obtained for the relationships of the rainfall characteristics to the overflow volumes at Site II revealed that the coefficient of 0.596, for the relationship of total overflow volume to total rainfall was the most significant.

The resulting linear regression equation is:

$$V_2 = 31770R - 8879$$

where V_2 = total overflow volume at Site II, cu m
R = total rainfall, cm

This equation predicts that it requires 0.28 cm (0.11 in.) of rain to cause the combined sewer to overflow at Site II, and that a volume of 8070 cu m (2,132,000 gal.) can be expected at Site II for each 0.25 cm (0.10 in.) of rain that falls after the first 0.28 cm (0.11 in.).

Because overflows are caused at Site I by 0.20 cm (0.08) and at Site II by 0.28 cm (0.11 in.), there were a few storms during the project where Site I was operated to treat combined sewer overflows but no overflow occurred at Site II.

In addition to the volumes of overflow generated by rainfall events, the rate at which these overflow volumes arrive must be considered. These rates, when greater than plant capacities, will cause plant bypass to the Root River.

Regression analyses were run to determine the relationships, if any, between bypass occurring during site operation and the rainfall characteristics (including days since the last overflow event). The runs for which the necessary data was available are given in Table 20. The resulting correlation coefficients for the regression analyses are given in Table 21, below.

TABLE 21. CORRELATION COEFFICIENTS FOR REGRESSION ANALYSES PERFORMED ON PLANT BYPASS VOLUMES DURING OPERATION AND RAINFALL CHARACTERISTICS

	Plant bypass volume	
Independent variables	Site I	Site II
Total rainfall	0.616	0.648
Average intensity	0.306[a]	0.572
Total rainfall and average intensity	0.628	0.756
Total rainfall, average intensity, and days since last run	0.629	0.792
Total rainfall, average intensity, maximum intensity, and days since last run	0.643	0.792
Total rainfall, and maximum intensity	0.636	0.686

a. Not significant at the 99 percent level of confidence; all other values are significant at the 99 percent level of confidence.

The most significant correlation coefficients are 0.616 for the relationship between plant bypass volumes at Site I and total rainfall; and 0.756 for the relationship of plant bypass volumes at Site II to total rainfall and average rainfall intensity. Both these coefficients are significant above the 99 percent confidence level (14).

The resulting linear regression equations are:

$$BV_1 = 671R - 517$$
$$BV_2 = 3454R + 12737E - 5873$$

TABLE 20. RAINFALL CHARACTERISTICS AND TREATMENT SITE BYPASS VOLUMES

Run No.	Site I Bypass Volume During Run		Site II Bypass Volume During Run		Total Rainfall		Average Intensity		Maximum Intensity		Days Since Last Run Site I	Days Since Last Run Site II
	cu m	gal. x10^4	cu m	gal. x10^4	cm	in.	cm/hr	in./hr	cm/hr	in./hr		
8	0	0	0	0	0.46	0.18	0.31	0.12	0.56	0.22	10	10
14	0	0	0	0	2.54	1.00	0.33	0.13	0.57	0.22	13	13
18	0	0	0	0	1.68	0.66	0.16	0.06	2.03	0.80	14	14
19	0	0	0	0	1.45	0.57	0.10	0.04	0.18	0.07	15	15
20	0	0	0	0	1.35	0.53	0.16	0.06	0.30	0.12	4	4
21	280	7	0	0	2.39	0.94	0.30	0.12	1.14	0.45	14	14
22	4122	109	3596	95	3.12	1.23	0.22	0.09	0.56	0.22	20	20
24	341	9	1476	39	1.57	0.62	0.32	0.13	6.10	2.40	6	6
25	0	0	0	0	0.66	0.26	0.20	0.08	0.46	0.18	15	15
26	2120	56	24034	635	3.81	1.50	0.85	0.33	5.08	2.00	10	10
27	0	0	0	0	0.30	0.12	0.20	0.08	0.33	0.13	7	7
28	0	0	0	0	1.09	0.43	0.10	0.04	0.18	0.07	3	3
29	0	0	0	0	1.14	0.45	0.15	0.06	3.05	1.20	3	3
30	0	0	0	0	0.51	0.20	0.61	0.24	2.29	0.90	2	2
31	0	0	1514	40	1.32	0.52	0.33	0.13	1.22	0.48	1	1
32	366	10	11734	310	1.98	0.78	0.40	0.16	0.56	0.22	2	2
33	68	2	2271	60	0.53	0.21	0.14	0.06	3.66	1.44	0	0
34	1407	37	0	0	0.64	0.25	0.30	0.12	6.98	2.75	5	5
35	57	2	4542	120	1.68	0.66	0.12	0.05	3.43	1.35	0	0
36	0	0	0	0	0.53	0.21	0.12	0.05	2.29	0.90	16	16
37	49	1	0	0	1.52	0.60	0.28	0.11	3.31	1.50	0	16
38	2877	76	20818	550	2.59	1.02	0.52	0.20	3.81	1.50	3	3
39	0	0	0	0	0.41	0.16	0.41	0.16	5.72	2.25	2	2
40	121	3	2650	70	2.41	0.95	0.19	0.07	3.35	1.32	22	22
41	0	0	0	0	1.22	0.48	0.13	0.05	0.41	0.16	7	7
42	0	0	757	2000	0.79	0.31	0.79	0.31	3.25	1.28	12	12
43	0	0	0	0	0.33	0.13	0.12	0.05	0.69	0.27	3	3
44	0	0	0	0	0.51	0.20	0.20	0.08	1.07	0.42	16	16
45	0	0	0	0	1.42	0.56	0.24	0.09	2.29	0.90	6	22

where BV_1 = plant bypass volume at Site I, cu m
BV_2 = plant bypass volume at Site II, cu m
R = total rainfall, cm
E = average rainfall intensity, cm/hr

When these regression analyses were begun, it was expected that a rainfall intensity (average or maximum) would be necessary to develop a significant relationship. This situation was the case at Site II, but not at Site I. At Site I, the plant bypass during operation, like the total overflow volume, was related only to the total rainfall.

An explanation of this difference is the size of the sewers contributing to Site I. Through observations, it was found that these sewers were usually in a surcharged condition during a combined sewer overflow event (this surcharge is also predicted by the math model). Even in dry weather, the sewers were usually running full. In the surcharged condition, the rate of overflow could not change with corresponding changes in rainfall intensities because the rainfall runoff could not reach the site unimpeded.

The total overflow volumes monitored from May to September 1974 are presented in Table 22. The totals of 266,214 cu m (70,333,000 gal.) at Site I and 743,677 cu m (196,480,000 gal.) at Site II are the volumes of combined sewer overflow that would have discharged directly to the Root River had no treatment been attempted. The treatment of these overflows will be discussed later in this section.

The large volume of overflow at Site II in September was caused by the interceptor sewer being blocked with debris for six days causing 171,000 cu m (45,178,000 gal.) of dry weather flow to overflow.

It should be noted that combined sewer overflows occurred on 56 of a possible 153 days, and during the wet months of May and June on 35 of 61 days. Rainfalls causing overflow did not necessarily occur this often; instead, some large rainfalls caused the sites to experience overflow, especially Site I, for 1 to 3 days after the end of the rain storm. This overflow happened mainly during the wet months of May and June.

The values obtained for 1973 are separated from the values obtained for 1974 because during most of 1973, the systems were not set to start automatically when an overflow began. Instead, operating personnel traveled to the sites and manually placed the treatment systems in operation; sampling began at this time. Due to travel time, the first 45 to 60 minutes of the overflows were missed. A comparison of the two sets of data will reveal the effect of this problem on the parameter concentrations in the composite sample obtained for the runs.

As was expected, the quality of the combined sewer overflows varied widely from overflow to overflow. For each parameter listed, the value given represents the arithmetic mean of composite samples for the number of runs indicated (the only exception is the fecal coliforms; the value presented is the geometric mean).

TABLE 22. DAYS AND VOLUMES OF COMBINED SEWER OVERFLOWS IN MAY TO SEPTEMBER, 1974

Month	Days of overflow	Overflow volume Site I		Overflow volume Site II	
		cu m	gal.	cu m	gal.
May	19	163,414	43,174,000	333,194	88,030,000
June	16	76,502	20,212,000	181,566	47,970,000
July	7	19,209	5,075,000	31,264	8,260,000
August	4	6,938	1,833,000	22,256	5,880,000
September	10	151	39,894	175,397	46,340,000
TOTAL	56	266,214	70,334,000	743,677	196,480,000

Three important aspects of the quality characteristics will be omitted here, but will be covered in detail in Section VI, STORM WATER MANAGEMENT MODEL; they are:

1. Quality variations with time.
2. Discussion of the first flush phenomenon.
3. The relationship between quality characteristics and the rainfall characteristics.

A comparison of the means for 1973 and 1974 at Site II shows that the concentrations of the parameters more than doubled from 1973 to 1974; the probable reason: a change in the overflow sampling. In 1973, sampling was not begun until personnel arrived at the site. This delay meant that a significant amount of overflow was missed including the highly polluted "first flush" of the sewers, and therefore a final composite would not reflect the true pollutional strength of the combined sewer overflow event but some lower value. In 1974, automatic sampling began when the site turned on automatically; therefore, the composite sample was representative of the entire overflow. This sampling of the entire overflow event may explain the higher composite sample mean quality concentrations for 1974. This indicated that the pollutional strength of the first flush may be very significant at Site II.

A similar change in sampling technique occurred at Site I, but it is not reflected in a comparison of mean quality parameters between 1973 and 1974. This observation tends to indicate that the first flush effect is not as great at Site I as it is at Site II.

Table 23 presents a comparison of the BOD, TOC and SS concentration means obtained for 1971 and 1974 at Site I and Site II. The 1974 means at Site I are compared to two 1971 overflow means because these two overflows both contributed to Site I when site construction was completed. Using the "t" statistic for the comparison of two means (15) a significant difference, at the 95 percent level of confidence, was found between the parameters at Site I in 1974 and the Chatham and Dodge Streets overflow in 1971. A significant difference was also found between the BOD concentrations for Site II in 1974 and the Site II overflow discharge point in 1971.

The difference between the 1974 overflow quality at Site I and the Chatham and Dodge Streets overflow quality determined in 1971 before site construction could be due to three factors. The first factor is that a large section of the Chatham Street combined-sewer-drainage area was separated in 1972 which greatly reduced the volumes of storm water discharging at the Chatham and Dodge Streets overflow point. This reduction of storm water flow would also reduce the significance of any first flush. The second factor is that 14 percent of the overflow at Chatham and Dodge Streets is diverted to Site II by a flow splitting device in the sewer. These two factors would tend to decrease the impact of the high pollutional strength Chatham and Dodge Streets overflow on the composite value for the Michigan and Dodge plus Chatham and Dodge Streets overflows taken from the Site I

TABLE 23. COMPARISON OF MEANS FOR 1971 AND 1974 OVERFLOW QUALITY

Site	Parameter	Mean, mg/l 1971	Mean, mg/l 1974	Significant difference at 95% Confidence Level
I	BOD	79[a]	93	no
		212[b]		yes
	TOC	98[a]	95	no
		194[b]		yes
	SS	299[a]	266	no
		669[b]		yes
II	BOD	212	110	yes
	TOC	238	122	no
	SS	443	661	no

a. Michigan and Dodge Streets overflow (overflow No. 1, Figure 1).
b. Chatham and Dodge Streets overflow (overflow No. 3, Figure 1).

wetwell in 1974. The third factor, and possibly the most important, is the length of the sampling periods. During 1971, the automatic samplers often plugged or malfunctioned after one or two hours of sampling; therefore, the samples taken probably covered the first flush only. In 1974, the entire overflow at Site I was sampled and as previously discussed, the overflows were usually of long durations. The composite sample, then, would be diluted by the low pollutional strength samples taken after long periods of continuous overflow.

The BOD concentration at Site II was significantly lower in 1974 than 1971. In addition, there were changes, although not statistically significant, in the TOC and SS concentrations. The TOC concentration decreased and the SS concentration increased. The decrease in BOD and TOC concentrations might have been due to the same change in sampling techniques that affected the mean concentrations at Site I between 1971 and 1974. This, however, does not explain the increase from 1971 to 1974 in the mean SS concentrations. Possible factors cited are: changes in the drainage area, changes in dry weather flow rates, and/or addition of inorganic solids to the sewer system, thereby increasing the SS concentrations but not the BOD and TOC concentrations.

Table 24 gives a comparison of the BOD, TOC, SS and total phosphorus concentration means for Site I and Site II based on the sampling done in 1974. The only significant difference in the overflow quality between the two sites is in the SS concentrations.

TABLE 24. COMPARISON OF MEANS FOR SITES I AND II OVERFLOW QUALITY (1974)

Parameter	Mean, mg/l Site I	 Site II	Significant difference at 95% Confidence Level
BOD	93	110	no
TOC	95	122	no
SS	266	661	yes
Total P	3.13	2.83	no

The difference in SS concentrations is due to the difference in the contributing areas and the conditions of the contributing sewers during dry weather. The difference in the contributing areas would account for the greater SS concentrations at Site II because the total gutter length for the area that contributes to Site II is greater than 610 m (2000 ft) while the total gutter length that contributes directly to Site I is only 38 m (125 ft). A good indication of the effect of this difference is the percentage of the SS concentrations that are nonvolatile. On the average the SS at Site II are 73.1 percent nonvolatile and at Site I the SS are 49.6 percent nonvolatile. Therefore, Site II is receiving large quantities of inorganic solids, probably due to much larger volumes of surface runoff than at Site I.

The dry weather flow in the sewer system is also important. As observed during the project and predicted by the Storm Water Management Model, the sewers contributing to Site I were usually running near capacity during dry weather while the sewers contributing to Site II were running at rates much lower than capacity. The rates in the Site I sewers would prevent large deposition of solids during dry weather conditions and when wet weather flow conditions began, the scouring effect of the increased flows would be significantly less than at Site II. Conversely, this difference would indicate a much greater first flush pollutional load at Site II than at Site I.

These differences in the contributing areas and combined sewer systems would also be responsible for the larger pollutional strength of the Site II overflow in 1971 when it is compared to the pollutional strength of the Michigan and Dodge Streets overflow discharge (Table 23).

Table 25 presents a comparison between overflow quality characteristics (BOD, SS and total phosphorus) as found in Racine and 12 other cities

TABLE 25. COMPARISON OF QUALITY OF COMBINED SEWER OVERFLOWS FOR RACINE AND VARIOUS OTHER CITIES(9)[a]

City	Years	BOD, mg/l	SS, mg/l	Total phosphorus, mg/l as P
Racine (Site I)	1974	93	266	3.13
Racine (Site II)	1974	110	661	2.83
Berkeley, CA	1968-69	60	100	--
Brooklyn, NY	1972	180	1051	--
Bucyrus, OH	1968-69	120	470	3.5
Cincinnati, OH	1970	200	1100	--
Des Moines, IA	1968-69	115	295	11.6
Detroit, MI	1965	153	274	4.9
Kenosha, WI	1970	129	458	5.9
Milwaukee, WI	1969	55	244	--
Roanoke, VA	1969	115	78	--
Sacramento, CA	1968-69	165	125	--
San Francisco, CA	1969-70	49	68	--
Washington, D.C.	1969	71	622	1.0
Average for other 12 cities		118	407	5.4

a. Since different sampling methods, number of samples, and other procedures were used, the data presented here are for general comparison only.

where combined sewer overflow studies were conducted (16).

The BOD concentration in the Racine combined sewer overflow is similar to the average value found in the 12 other studies. The SS concentration is less than the average at Site I and more than the average at Site II. The total phosphorus concentrations for both Site I and Site II are less than the average for the five cities that determined total phosphorus during their studies.

On the whole, the values obtained for the combined sewer overflow quality in Racine are in agreement with the patterns established by the many studies conducted on the quality of combined sewer overflows.

Storm Sewer Discharges - The quality determinations for storm sewer discharges are based on the data collected from Site IIA. The wetwell for the site is the discharge of a storm water collection system servicing a 6.3 ha (15.6 acre) area.

The volumes of storm water discharge at Site IIA will not be discussed in detail because of the previously covered problems encountered with the circular flow recording chart and the volume totalizer.

The means, ranges, and 95 percent confidence intervals for the measured storm water quality parameters are given in Table 26. The quality characteristics are based on the samples collected in 1973 and 1974 because during the entire duration of the project, Site IIA was set to start automatically when a discharge occurred. Therefore, sampling of the discharge began immediately and continued until the discharge ceased.

TABLE 26. 1973 AND 1974 STORM WATER CHARACTERISTICS - SITE IIA

Parameter	Mean[a] concentration mg/l	Range, mg/l	No. of events
BOD	15 ± 5	3 - 64	30
TOC	46 ± 13	7 - 180	31
Suspended solids	376 ± 122	55 - 1,400	31
Total phosphorus (as P)	0.37 ± 0.12	0.10 - 1.65	29
Fecal coliform density	580[b]	0 - 21,000	30
pH	--	6.90 - 8.35	29

a. Limits given for mean are for 95 percent confidence interval using "t" distribution. Means given are arithmetic except for fecal coliform density which is geometric.

b. Number/100 ml.

The quality characteristics for the storm sewer discharge in 1971 were previously presented in Table 7. A comparison of the mean values using the "t" statistic reveals that there is a significant decrease in the BOD concentrations between 1971 and 1973-74. Similarily, there is a decrease, although not statistically significant, in the TOC and SS concentrations.

Comparison of Quality Characteristics of
Storm Sewer Discharge for 1971 and 1973-74

Parameter	1971 Mean, mg/l	1973-74 Mean, mg/l	Significant at the 95 percent confidence level
BOD	39	15	Yes
TOC	51	46	No
SS	445	376	No

The decreases might be due to better street cleaning practices or other changes that occurred in the storm water drainage area.

Table 27 gives a comparison of the storm sewer discharge quality determined in Racine and the quality determined in nine other cities that conducted storm sewer discharge studies (18).

TABLE 27. COMPARISON OF QUALITY OF STORM SEWER
DISCHARGES FROM RACINE AND VARIOUS OTHER CITIES (9)[a]

City	Years	BOD, mg/l	SS mg/l
Racine, WI	1973-74	15	376
Ann Arbor, MI	1965	28	2080
Des Moines, IA	1969	36	505
Los Angeles, CA	1967-68	9	1013
Madison, WI	1970-71	--	81
New Orleans, LA	1967-69	12	26
Roanoke, VA	1969	7	30
Sacramento, CA	1968-69	106	71
Tulsa, OK	1968-69	11	247
Washington, D.C.	1969	19	1697
Average for 9 cities, not incl. Racine		28	639

a. Since different sampling methods, number of samples, and other procedures were used, the data presented here are for general comparison only.

The storm sewer discharge in Racine is less than the average of the pollutional strengths found in the nine other cities. The values vary widely, however, because they are dependent on many factors: land use, average days between rainfalls, street sweeping practices, etc. Therefore, it is not possible to definitely state why the Racine storm sewer discharge is lower in pollutional strength than many other cities. It is probably a combination of factors that affect the rainfall runoff and the deposition of solids in the drainage area during dry weather.

Regression analyses (13) were run on the storm sewer discharge quality characteristics (BOD, SS, and fecal coliform concentrations) in an attempt to relate them to the rainfall characteristics.

Over the two years of the project, Site IIA operated 32 times, but because of operational problems or problems with the automatic samplers only 21 runs were used for these analyses. Table 28 lists the data obtained from these 21 runs.

TABLE 28. RAINFALL CHARACTERISTICS AND STORM SEWER DISCHARGE QUALITY (SITE IIA)

Run No.	BOD, mg/l	SS, mg/l	Fecal coliform, no./100 ml	Average rainfall intensity, cm/hr	in./hr	Maximum rainfall intensity cm/hr	in./hr	Days since last run
5	6	55	1	0.46	0.18	1.02	0.40	17
6	3	105	1	0.25	0.10	2.44	0.96	2
13	5	159	17000	1.68	0.66	16.00	6.30	42
14	6	423	70	0.33	0.13	0.57	0.22	3
16	5	76	4800	0.33	0.13	1.90	0.75	3
17	10	383	3500	0.58	0.23	3.61	1.42	4
18	15	514	500	0.16	0.06	2.03	0.80	15
19	64	156	310	0.10	0.04	0.18	0.07	15
20	16	102	720	0.16	0.06	0.30	0.12	4
21	22	171	380	0.30	0.12	1.14	0.45	15
22	4	128	240	0.22	0.09	0.56	0.22	21
25	19	223	1	0.20	0.08	0.46	0.18	5
27	15	70	3300	0.20	0.08	0.33	0.13	8
28	9	160	330	0.10	0.04	0.18	0.07	3
29	24	841	450	0.15	0.06	3.05	1.20	3
30	13	574	390	0.61	0.24	2.29	0.90	2
31	7	137	410	0.33	0.13	1.22	0.48	1
32	6	121	470	0.40	0.16	0.56	0.22	2
34	23	660	800	0.30	0.12	6.98	2.75	5
35	17	299	710	0.12	0.05	3.43	1.35	1
39	15	304	13300	0.41	0.16	5.72	2.25	2

The BOD, SS, and fecal coliform concentrations are the values obtained from the composite sample for the duration of the storm sewer discharge. The number of days since the last run is the number since a rainfall of sufficient volume or intensity occurred that could cause storm sewer discharge.

The correlation coefficients for the different relationships are given in Table 29. The only statistically significant correlations (17) were obtained between the fecal coliform concentrations and the rainfall characteristics, and the most significant was the relationship between the fecal coliform concentration and the maximum rainfall intensity, which resulted in a correlation coefficient of 0.808. The resulting linear regression equation is:

$$N = 1017 - 343$$

where N = fecal coliform concentration, No./100 ml
M = maximum rainfall intensity, cm/hr

TABLE 29. CORRELATION COEFFICIENTS FOR REGRESSION ANALYSES PERFORMED ON STORM SEWER DISCHARGE QUALITY & RAINFALL CHARACTERISTICS

Dependent variable	Independent variable (x_1)	Independent variable (x_2)	Correlation coefficient
BOD	Average intensity		-0.313
BOD	Maximum intensity		-0.138
BOD	Days since last run		0.027
BOD	Average intensity	Days since last run	0.444
BOD	Maximum intensity	Days since last run	0.190
Suspended solids	Average intensity		-0.061
Suspended solids	Maximum intensity		0.197
Suspended solids	Days since last run		-0.265
Suspended solids	Average intensity	Days since last run	0.276
Suspended solids	Maximum intensity	Days since last run	0.456
Fecal coliforms	Average intensity		0.753[a]
Fecal coliforms	Maximum intensity		0.808[a]
Fecal coliforms	Days since last run		0.492[b]
Fecal coliforms	Average intensity	Days since last run	0.753[a]
Fecal coliforms	Maximum intensity	Days since last run	0.809[a]

a. Significant at the 99 percent confidence level.

b. Significant at the 95 percent confidence level.

After obtaining this relationship, another regression analysis was made using the log of the fecal coliform concentration and the maximum rainfall intensity. The resulting correlation coefficient for this relationship was only 0.440.

Removal of Pollutants from Storm Generated Discharges

Efficiency of Drum Screens - Tables 30, 31, and 32 present summaries of the data on removal of pollutants by the drum screens for Sites I, II and IIA, respectively. Since only screening was used at Site IIA, these removal percentages and effluent characteristics are the total removal percentages and final effluent for the process. Raw data on the operation of the drum screens is presented in Appendix IV-C on a run-by-run basis. The results of laboratory analyses on the screen effluent samples are presented on a run-by-run basis in Appendix IV-D.

All data collected are used in the data summaries. It should be noted that during 1973, the procedure was to collect samples manually at a predetermined time interval, usually 20 min. During 1974, automatic samplers were used. They were set to take a sample every 10 min and were started when personnel arrived at the sites.

The average removals of SS from the combined sewer overflows by the drum screens at Sites I and II were 32 and 36 percent, respectively. The percent removals, however, varied widely from run to run and, therefore, the arithmetic mean of the percent removals was not used.

The average hydraulic loading at Site I was 1.81 cu m/min/sq m (44.4 gpm/sq ft) with individual runs ranging from 0.70 to 3.50 (17.2 to 85.9). The average solids loading was 59.8 kg/1,000 sq m (12.2 lb/1,000 sq ft) with run values ranging from 8.40 to 224 (1.72 to 45.8). At Site II, the average hydraulic loading was 1.39 cu m/min/sq m (34.1 gpm/sq ft) with a range of 0.39 to 3.67 (9.6 to 90.1). The average solids loading was 66.0 kg/1,000 sq m (13.5 lb/1,000 sq ft) and the values ranged from 8.87 to 246 (1.8 to 50.3).

Although the solids loadings are similar at both sites, the hydraulic loading at Site I is greater than at Site II. This may be the reason why the screening process achieved only 32 percent SS removal at Site I while achieving 36 percent SS removal at Site II.

Screen backwash characteristics are presented in Table 33 for 1973. Screen backwash samples were not taken in 1974 for reasons previously explained. Based on the mean values for the samples collected, the backwash water at Site I is 0.18 percent solids and the solids are 70 percent volatile. The backwash water at Site II is 0.28 percent solids and the solids are 50 percent volatile. These results, as well as personal observations, indicate that Site II was receiving much larger quantities of inorganic solids than Site I during 1973. Sewer construction in the drainage area may account for the increased loadings.

TABLE 30. SCREENED EFFLUENT CHARACTERISTICS AND AVERAGE PERCENT REMOVALS BY THE SCREENS - SITE I

Parameter	Screen effluent mean concentration,[a] mg/l	Range	Average percent removal[b]	No. of events
BOD	66 ± 9	16 - 169	28	40
Dissolved BOD	21 ± 5	5 - 36	0	18
TOC	61 ± 8	14 - 147	38	39
Dissolved organic carbon	20 ± 4	8 - 39	5	19
Suspended solids	191 ± 33	18 - 630	32	40
Suspended volatile solids	66 ± 26	2 - 142	55	11

TABLE 31. SCREENED EFFLUENT CHARACTERISTICS AND AVERAGE PERCENT REMOVALS BY THE SCREENS - SITE II

Parameter	Screen effluent mean concentration,[a] mg/l	Range	Average percent removal[b]	No. of events
BOD	50 ± 13	12 ± 131	42	26
Dissolved BOD	26 ± 10	2 - 82	0	16
TOC	50 ± 11	10 - 115	44	25
Dissolved organic carbon	23 ± 6	5 - 61	12	18
Suspended solids	329 ± 101	65 - 1098	36	26

TABLE 32. SCREENED (FINAL) EFFLUENT CHARACTERISTICS AND AVERAGE PERCENT REMOVAL BY THE SCREEN - SITE IIA

Parameter	Screen effluent mean concentration,[a] mg/l	Range	Average[b] percent removal	No. of events
BOD	12 ± 4	2 - 34	20	27
TOC	27 ± 6	7 - 65	41	28
Suspended solids	187 ± 52	40 - 513	50	29
Total phosphorus (as P)	0.22 ± 0.05	0.05 - 0.53	40	27
Fecal coliform density[c]	450	0 - 48,000	22	28
pH	--	6.90 - 8.40	--	28

a. Limits given for mean are for 95 percent confidence interval using "t" distribution. Means given are arithmetic except for fecal coliform density which is geometric.

b. Average removal calculated as [(mean raw - mean eff.)/mean raw] x 100.

c. Number/100 ml.

TABLE 33. SCREEN BACKWASH CHARACTERISTICS (1973)
(mg/l)

	Site I				Site II			
Run No.	Total solids	Total Volatile solids	Suspended solids	Suspended volatile solids	Total solids	Total Volatile solids	Suspended solids	Suspended volatile solids
3	4,500	3,250	3,700	2,670				
4			2,500	2,100				
5	1,100	684	822	507				
6	2,750	1,960	2,640	2,090	3,370	2,030	4,140	2,430
7	1,711	1,177	1,990	1,735				
8	2,684		3,414	2,459				
10	2,185		1,715	1,270				
11	851		675	393				
12	571		351	187	2,800		4,670	2,790
13	461		276	116	4,385	1,600	3,755	1,346
14	867		713	504	1,968		1,962	795
15	1,108		714	431				
16	1,079		776	553				
17	769		495	304				
18	8,671		7,888	4,584				
19	2,098		1,475	1,090	2,242		1,110	564
20	1,464		1,194	912	1,356		966	408
Mean	2,054	1,768	1,843	1,289	2,687	1,815	2,767	1,389

For the Site I screens, an average volume of 2.4 percent of the total plant flow was used for backwashing the screens at a head loss of from 30 to 41 cm (12 to 16 in.). At Site II, screen backwash requirements averaged 3.1 percent of the total plant flow using the same head loss as at Site I to initiate backwashing. Backwash requirements varied with hydraulic loading and the solids loading on the drum screens during the run, with the frequency of backwashing being the greatest during a combination of high hydraulic and solids loadings. For the Site I drum screens, backwashing was continuous during periods of high flow, but intermittent when the flow rate dropped below 80 percent of the design capacity. Backwashing was generally intermittent at Site II. It appears that the screen backwashing requirements are greater for Site II than Site I because of the high flow - short duration nature of the overflows at Site II.

A mass balance was performed on the screening process using the average values found for the raw flow SS concentrations, volumes treated, backwash water SS concentrations, and backwash water volumes used. For Site I, the mass balance predicted a screened effluent SS concentration of 221 mg/l compared to the 95 percent confidence interval of 158 to 224 mg/l obtained from the sampling program. For Site II, the mass balance predicted a screened effluent SS concentration of 574 mg/l compared to the 95 percent confidence interval of 228 to 430 mg/l obtained from the sampling program.

Seventeen runs were selected during which no major problems were encountered with the drum screens or the sampling program. Regression analyses (13) were run on these selected runs in an attempt to establish relationships between the SS removals achieved by the drum screens and the hydraulic and solids loadings. The following correlation coefficients were obtained for percent SS removals at Sites I and II. Regression analysis was not performed on the Site IIA data due to the extreme scatter of the data.

Site	Dependent Variable	Correlation Coefficient
I	Solids loading	0.154
	Hydraulic loading	0.101
	Solids and hydraulic loading	0.155
II	Solids loading	0.335
	Hydraulic loading	0.052
	Solids and hydraulic loading	0.343

None of the obtained correlation coefficients is statistically significant and, therefore, no linear relationship could be established for percent SS removals and the hydraulic and solids loadings.

At Site IIA, storm water is treated by screening only and then discharged to the Root River. An average of 50 percent of the influent SS was removed by the screen, although the percent removals varied widely for individual runs.

The average hydraulic loading for the Site IIA drum screen was

1.34 cu m/min/sq m (32.9 gpm/sq ft) with individual storm values ranging from 0.68 to 2.42 cu m/min/sq m (16.7 to 59.4 gpm/sq ft). The average solids loading was 71.4 kg/1,000 sq m (14.6 lb/1,000 sq ft) and the solids loading ranged from 12.1 to 227.8 cu m/min/sq m (2.5 to 46.6 lb/1,000 sq ft). The average screen backwash requirement was 12.5 percent of the raw flow. This value is high because of a process modification: early in the project, the differential pressure switch that controlled the backwash pump was disconnected and the pump was rewired so that it washed the screen after every spiral screw pump operation. This change resulted in much more frequent backwashing of the screen.

Efficiency of Screening/Dissolved Air-Flotation - Data on the operation of the flotation system is given on a run-by-run basis in Appendix IV-C. The results of laboratory analyses of the final effluent samples is given on a run-by-run basis in Appendix IV-D. All of the data collected during the project are used in the data summaries although results from some individual runs may have been affected by operational problems and varying chemical dosages.

The operational parameters for the flotation process may be summarized as follows:

Site	Overflow rate, cu m/min/sq m (gpm/sq ft)		Detention time, hr		Pressurized flow rate, cu m/min (gpm)	
	Mean	Range	Mean	Range	Mean	Range
I	0.92 (2.25)	.045 - .172 (1.10 - 4.22)	0.50	0.23 - 0.91	3.21 (858)	1.92 - 4.35 (513 - 1162)
II	.075 (1.84)	.019 - .22 (0.47 - 5.40)	0.76	0.18 - 2.12	2.59 (692)	1.42 - 4.16 (379 - 1111)

The pressure in the pressurization tank averaged 2.86 kg/sq cm g (41.2 psig), the skimmer flight speed was 0.74 m/min (2.4 gpm), and the skimmers were on for two minutes and then off for 12 minutes. The pressurized flow rate at Site I averaged 47.7 percent of the raw flow rate and at Site II the pressurized flow rate averaged 45.8 percent of the raw flow rate (assuming all the flotation tanks to be in operation and therefore all pressurized flow pumps in use).

From this summary of operating data, it should be noted that the Site II surface overflow rate is less than Site I and the Site II flotation tank detention time is greater than Site I. These factors may be responsible for better removals that were achieved at Site II.

Floated sludge characteristics are presented in Table 34. At Site I the floated sludge ranged from 1.4 to 10.0% solids with a mean value of 5.1% solids. The volatile content of the sludge SS averaged 45.1%. At

TABLE 34. FLOATED SLUDGE CHARACTERISTICS (1973)
(mg/l)

Run No.	Site I Total solids				Site II Total solids			
	Total solids	Total Volatile solids	Suspended solids	Suspended volatile solids	Total solids	Total Volatile solids	Suspended solids	Suspended volatile solids
3	55,600	30,400	54,100	28,900	--	--	--	--
4	--	--	25,800	14,900	--	--	--	--
5	79,900	30,800	74,600	27,800	--	--	--	--
6	44,400	21,600	38,800	17,950	54,600	16,600	56,650	17,200
7	178,400	69,200	--	--	--	--	--	--
8	78,166	43,092	47,350	22,575	--	--	--	--
9	109,688	36,639	100,200	36,750	--	--	--	--
10	98,300	52,900	--	--	--	--	--	--
11	16,590	9,940	13,700	7,770	--	--	--	--
12	57,200	21,200	50,400	17,300	9,930	3,360	8,180	2,620
13	99,994	44,743	78,841	34,875	4,385	1,600	3,755	1,346
14	42,922	22,602	38,583	19,958	53,734	14,141	51,100	13,567
15	45,218	22,843	43,429	20,621	97,927	25,695	93,533	25,667
16	28,243	18,376	--	--	--	--	--	--
17	50,750	23,968	49,050	22,884	--	--	--	--
18	51,634	21,707	47,364	26,500	48,942	11,179	42,950	9,190
19	34,340	25,180	--	--	83,828	20,141	--	--
20	58,478	35,956	--	--	56,137	16,778	34,000	10,150
Means	66,960	31,240	50,940	22,980	51,190	13,690	41,450	11,390

Site II, the floated sludge ranged from 0.4 to 9.4% solids with a mean value of 4.1% solids. The sludge SS volatile content averaged 27.5%. The percent volatile suspended solids is greater for the Site I sludge than the Site II sludge; this same condition was found for the drum screen backwash water. This also indicates that Site II receives much larger quantities of inorganic solids than Site I.

Frequently, the sludge volumes (backwash water plus floated sludge) exceeded the capacity of the sludge holding tanks; therefore, accurate measurements of the total sludge volumes produced by the system could not be obtained. Also, because the volume of floated sludge produced was to be calculated as the difference between the total sludge volume produced minus the backwash water volume used, the floated sludge volume could not be obtained. Instead, these volumes were estimated from a mass balance on the system later in this section.

Tables 35 and 36 present the final effluent quality characteristics and the average percent removals achieved for Site I and Site II, respectively. The removal efficiencies obtained for each parameter are for the combination of the screening and dissolved-air flotation processes. Seven time series studies were also conducted during the course of the project (both raw and effluent samples were taken). The results of these studies are presented and discussed in Section VI of this report, STORM WATER MANAGEMENT MODEL.

The calculation of the average percent removal by taking the arithmetic average of the individual percent removals resulted in negative values for dissolved BOD and dissolved organic carbon removals at Site I and dissolved BOD removal at Site II. The mean concentrations (mg/l) for these parameters through the system were:

Site	Parameter	Raw	Screened	Final
I	Dissolved BOD	22	21	24
	Dissolved organic carbon	22	20	21
II	Dissolved BOD	30	26	23

Using these values, the average percent removal results were calculated to be: 4.5 percent for dissolved organic carbon at Site I, and 23.3 percent for dissolved BOD at Site II. The dissolved BOD at Site I, however, still shows an increase in concentration. Since the dissolved-air flotation process is not expected to remove dissolved pollutants, it may be assumed, despite some variation, that no removal of the above pollutants was achieved.

Although the differences are not statistically significant, the data for Site I reveals that dissolved BOD and dissolved organic carbon may have

TABLE 35. FINAL EFFLUENT CHARACTERISTICS AND AVERAGE PERCENT REMOVAL - SITE I

Parameter	Effluent mean concentration,[a] mg/l	Range	Average[b] percent removal	No. of events
BOD	40 ± 7	12 - 148	50.1	44
Dissolved BOD	24 ± 3	6 - 49	-23.1	43
TOC	38 ± 5	16 - 96	47.1	44
Dissolved organic carbon	21 ± 2	9 - 39	- 3.6	44
Total solids	443 ± 28	286 - 725	25.7	43
Suspended solids	94 ± 19	6 - 353	59.7	44
Volatile suspended solids	40 ± 9	0 - 168	64.7	44
Total phosphorus (as P)	1.66 ± 0.25	0.40 - 3.62	46.6	43
Fecal coliform density[c]	500	0 - 940,000	--	42
pH	--	6.60 - 7.90	--	44

a. Limits given for mean are for 95 percent confidence interval using "t" distribution. Means given are arithmetic except for fecal coliform density which is geometric.

b. Value given is arithmetic mean of percent removals for all runs.

c. No./100 ml.

TABLE 36. FINAL EFFLUENT CHARACTERISTICS AND AVERAGE PERCENT REMOVALS - SITE II

Parameter	Effluent mean concentration,[a] mg/l	Range	Average[b] percent removal	No. of events
BOD	24 ± 4	5 - 51	60.4	31
Dissolved BOD	23 ± 5	4 - 69	- 2.2	30
TOC	26 ± 4	7 - 51	50.4	32
Dissolved organic carbon	19 ± 3	7 - 33	6.0	31
Total solids	410 ± 29	264 - 577	37.6	30
Suspended solids	113 ± 21	22 - 242	66.1	32
Volatile suspended solids	29 ± 7	5 - 75	57.0	32
Total phosphorus (as P)	0.87 ± 0.44	0.15 - 1.88	60.3	32
Fecal coliform density[c]	700	0 - 390,000	--	29
pH	--	3.50 - 7.90	--	32

a. Limits given for mean are for 95 percent confidence interval using "t" distribution. Means given are arithmetic except for fecal coliform density which is geometric.

b. Value given is arithmetic mean of percent removals for all runs.

c. No./100 ml.

actually increased through the flotation tanks. One possible explanation for the increase in these dissolved pollutants is the presence of sludge on the bottom of the flotation tanks. If the sludge begins to digest during dry weather, it would produce some dissolved organics that would then be picked up by the wet weather flow through the tanks. It is felt that if the sludge can be removed between system runs, this problem may be eliminated. However, it is possible that the dissolved fraction of the wastewater increases through the system as indicated by the tests.

The minimum pH value of 3.50 at Site II (Table 36) results from problems encountered with the ferric chloride feed system. As discussed previously, the raw flow was frequently overdosed with ferric chloride because the operating personnel had no control over the gravity feed system. When overdosing did occur, it resulted in very low effluent pH values (3.5 to 5.5).

For the most part the treatment achieved at Site II was better than the treatment achieved at Site I because of two factors:

1. At Site II, the flotation tank surface overflow rate was 19 percent less than at Site I.

2. Correspondingly, the flotation tank detention time at Site II was 52 percent greater than at Site I.

These differences may be explained by the fact that on the average, Site I ran at 54 percent of its rated capacity while Site II ran at only 39 percent of rated capacity.

Table 37 presents a comparison of the average percent removals achieved during 1973 and 1974. The overall treatment improved in 1974. The

TABLE 37. PERCENT REMOVALS FOR 1973
COMPARED TO PERCENT REMOVALS FOR 1974

Parameter	Site I		Site II	
	1973	1974	1973	1974
BOD	42.7	57.5	52.8	65.4
TOC	43.0	51.2	39.2	64.7
Total solids	25.8	25.7	31.8	41.1
Suspended solids	57.1	62.2	56.0	73.3
Volatile suspended solids	62.6	66.8	37.7	70.9
Total phosphorus (as P)	43.7	49.3	46.8	70.0

improvement occurred because 1973 was a period of startup and shakedown for the system. Many major problems were encountered with the equipment and standard operating parameters were not yet established. Undoubtedly, these factors contributed to producing efficiencies that were not truly indicative of removal efficiencies that could be achieved by the screening/dissolved-air flotation process. The 1974 results, therefore, give a better indication of the pollution removals that may be expected from a well operating screening/dissolved-air flotation unit that is treating combined sewer overflows: 65 percent BOD removal and 73 percent SS removal.

Tables 38 and 39 give the average percent removals averaged at Site I and II, respectively, on a mass basis. The mass of pollutants in the influent and the effluent of the sites for each run are given in Appendix IV-G, Tables G1 to G12.

Calculation of the percent removals in this manner increased all of the values, as shown in these data:

Site	Parameter	Average percent removed	
		Arithmetic mean	Mass basis
I	BOD	50.1	62.4
	Total organic carbon	47.1	60.0
	Total solids	25.7	28.1
	Suspended solids	59.7	67.6
	Volatile suspended solids	64.7	73.6
	Total phosphorus	46.6	53.2
II	BOD	60.4	69.5
	Total organic carbon	50.4	66.6
	Total solids	37.6	47.2
	Suspended solids	66.1	69.8
	Volatile suspended solids	57.0	67.3
	Total phosphorus	60.3	62.4

The reason for this increase is that the overall treatment efficiency was usually better for long duration runs (large volumes treated) than for shorter duration runs (small volumes treated). Therefore, the mass removals are greater than the arithmetic mean which gives equal weight to each run without regard to the volumes treated.

The principal reason for this phenomenon lies in system start-up time required (30 to 45 min) after effluent flow began before a good quality effluent was achieved. Once this quality was established it remained fairly constant for the entire run, however, the effluent quality did decrease during this time when the flow significantly increased but the effluent in quality was still better than in the first 30 to 45 min. As the run duration increased, the effluent samples obtained during the first part of the run became a lesser fraction of the total

TABLE 38. PERCENT REMOVALS BASED ON MASS OF POLLUTANTS - SITE I

Parameter	Total mass influent, kg	Total mass influent, lb	Total mass effluent kg	Total mass effluent lb	Percent removed
BOD	28,790	63,410	10,840	23,875	62.4
TOC	30,670	67,615	12,260	27,035	60.0
Total solids	216,630	477,150	155,770	343,110	28.1
Suspended solids	97,530	215,010	31,570	65,540	67.6
Volatile suspended solids	46,590	102,710	12,300	27,120	73.6
Total phosphorus (as P)	1,114	2,455	522	1,150	53.2

TABLE 39. PERCENT REMOVALS BASED ON MASS OF POLLUTANTS - SITE II

Parameter	Total mass influent,		Total mass effluent,		Percent removed
	kg	lb	kg	lb	
BOD	19,560	43,120	5,970	13,161	69.5
TOC	20,830	46,605	6,965	15,355	66.6
Total solids	211,399	466,050	111,690	246,232	47.2
Suspended solids	126,191	278,200	38,091	83,975	69.8
Volatile suspended solids	28,072	61,888	9,193	20,267	67.3
Total phosphorus (as P)	501.4	1,105	188.6	415.8	62.4

composite sample. Conversely, when the run was short, the samples taken during the first 30 to 45 min were a large fraction of the final composite. For that reason, the percent removals for short runs were usually less than for long runs.

Because of the long duration of the overflows at Site I, the site was run continuously for as long as 41 hours. This duration was achieved by allowing the site to run on its own during the low flow periods after the rainfall had ceased and the overflow subsided resulting in the site running unattended for up to 17 hours.

Figure 47 presents a plot of the treatment achieved at Site I (percent BOD removed) versus the run duration for 16 runs in 1974 that were of duration less than 780 min. The resulting plot reveals the trend of improving percent removals as run duration increases. The percent removal, based on the composite effluent sample, increases significantly with time until the 16-run average of 59 percent removed, is reached after 180 min.

Plant flows and effluent quality are plotted versus time and discussed in more detail in Section VI of this report, STORM WATER MANAGEMENT MODEL.

As stated previously, the treatment results are based on all of the runs despite variations in chemical additions, which were due to:

- Changes in the desired dosages based on bench scale flotation tests.
- Malfunctions of the chemical feed systems.

In an attempt to establish what effect chemical dosages had on treatment efficiency, the percent SS removals at Site II were compared to the corresponding ferric chloride dosages, as shown in the following table:

Ferric chloride dose (mg/l)	0	1-10	11-20	21-50	51-70	>70
Mean percent removal	47.2	71.0	70.6	82.2	71.0	71.5
Number of runs considered	5	3	7	5	6	4

Use of the "t" statistic for the comparison of two means (15) revealed that the only significant difference was between the percent SS removals for additions of zero ferric chloride and 21 to 50 mg/l. No other statistically significant differences were found, mainly because of the small sample sizes available for analysis. It appears that the ferric chloride dosage ranges of 1 to 20 and 51 to >70 mg/l all resulted in equal SS removals, namely 71 percent. The rate of 21 to 50 mg/l improved the removal to 82 percent, which corresponds with the bench scale results that predicted the best flotation at a ferric chloride addition of 40 to 50 mg/l.

Using average values obtained during the course of the project and the entire treated flow, a mass balance was performed on the entire treatment system as shown in the following table.

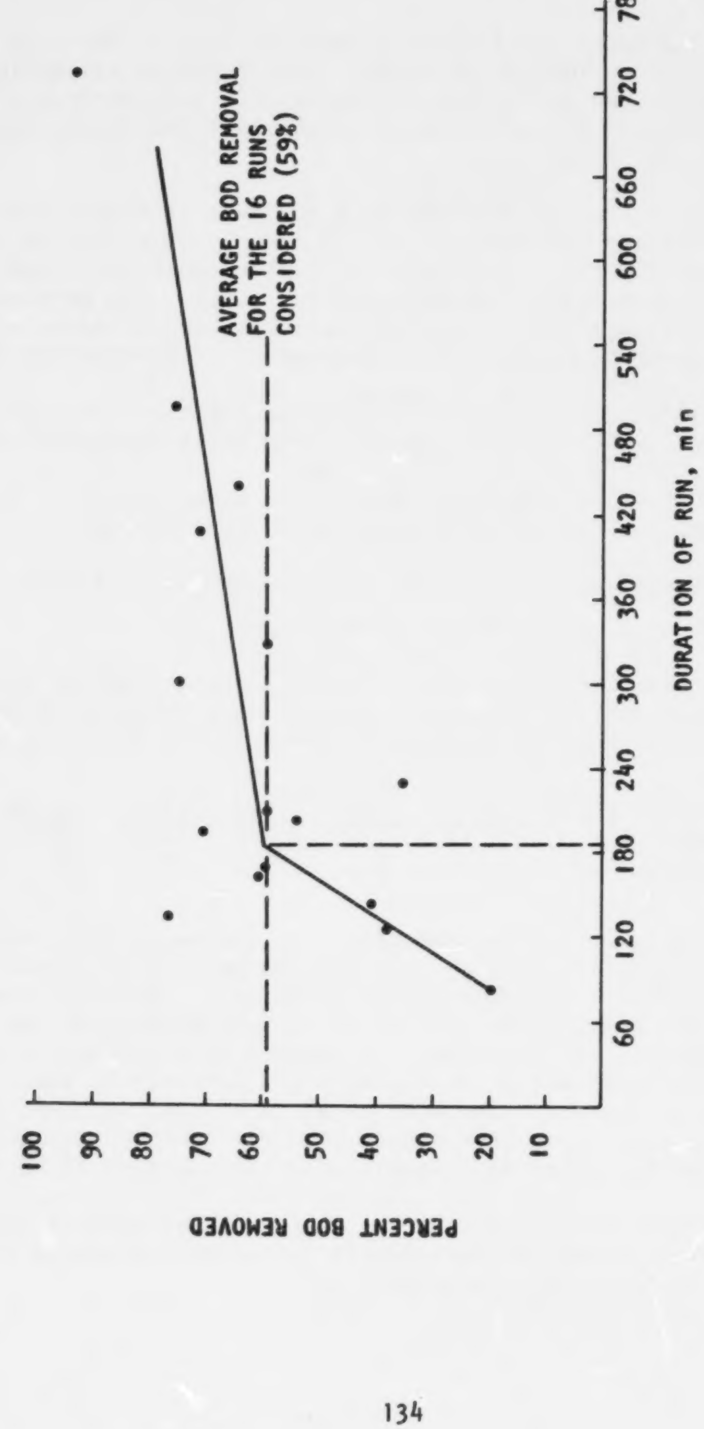


Figure 47. Percent BOD removal versus duration of run (Site 1).

Mass balance data

	Suspended solids (mg/l)				Volumes (cu m)	
Site	Influent	Effluent	Backwash	Floated sludge	Treated discharge	Backwash
I	266	94	1843	50940	8556	207
II	661	113	2767	41450	9572	301

The value used for the Site II influent SS concentration was the 1974 mean because the first flush portion of the combined sewer overflow was frequently missed in 1973, and, therefore, the composite samples did not give a true indication of the solids entering the site. From the mass balance, the following estimates on sludge production during the evaluation period were made:

Estimates of sludge production

	Site I	Site II
Volume floated sludge, cu m (gal.)	21.4 (5,641)	106.4 (28,108)
Total sludge volume, cu m (gal.)	228.4 (60,343)	407.4 (107,635
Total sludge suspended solids, %	0.64	1.29
Volume backwash/total sludge, %	91	74
Sludge production/vol treated, cu m (gal.)/1000 cu m (gal.)	26.7 (26.7)	42.6 (42.6)

It must be noted that all calculations assumed that no settling of solids occurred, but in actuality some solids did settle to the bottom of the flotation tanks.

The Site II values of volume of floated sludge produced, percent SS of the total resulting sludge, and sludge-volume-produced/volume-treated are all higher than Site I because of the very high concentrations of SS in the influent. The backwash water is a larger portion of the total sludge volume at Site I because of the higher hydraulic loadings on the drum screens at Site I causing the backwash pump to start more frequently and remain on longer than the backwash pump at Site II.

Efficiency of Chlorination - The concentrations of fecal coliform bacteria in the effluent from Sites I and II are summarized in Tables 35 and 36. The chlorination control was generally set to maintain a dosage of 10 to 15 mg/l of chlorine based on the incoming flow rate. It was difficult, however, to maintain this rate for the duration of a run because of the frequent loss of the chlorinator operating vacuum due to plugging of the injector with solids.

The geometric means for 42 runs at Site I and 29 runs at Site II (Site I: 500 colonies/100 ml; Site II: 700 colonies/100 ml) are both above the standard (200 colonies/100 ml) set for the Root River by the Wisconsin Department of Natural Resources (11). The geometric means take into account nine runs at Site I and eleven runs at Site II when no chlorine was added or when chlorine was added but no chlorine appeared in the final effluent because frequent plugging of the chlorine injector. A comparison of runs at Site I and Site II when no chlorine was added and when chlorine was added in sufficient quantities to produce an effluent residual follows:

	No. of events	Coliform geometric mean (No./100 ml)
No chlorine addition	14	54,000
Chlorine addition with effluent residual	37	113

Thus, whenever sufficient operating vacuum could be maintained during a run to produce a chlorine residual, the effluent fecal coliform counts were below the standard set by the Wisconsin Department of Natural Resources. This condition was achieved by a chlorine residual in the effuent of approximately 0.5 mg/l. The residence of the wastewater in the flotation tanks appears to be sufficient in terms of manner and time of chlorine contact.

Considering the highly variable concentration of fecal coliforms in the raw waste, secondary control of chlorination based on the residual in the effluent should be considered for combined sewer overflow treatment facilities, especially in light of the growing concern over the toxic effects of excessive chlorine residuals on the fauna in the receiving body. Standard operating conditions produced effluent residuals of 0.4 to 10.0 mg/l for the runs in which an effluent residual was detected.

Impact of Treatment on the Quality of Discharge to the Root River

One of the main objectives of the project is to show an improvement in the quality of the receiving water as a result of treatment of storm generated discharges in the test reach. The crucial elements in achieving this objective are to capture as much of the overflow as possible and to give it the best treatment possible before discharge to the Root River. The percent removals presented previously are the removals achieved through the treatment processes only; no consideration is given to pollution occurring from plant bypass.

The mass loadings to the Root River due to bypass volumes and site effluent volumes at Sites I and II in 1974 are given on a run-by-run basis in Appendix IV-H, Tables H-1 to H-12. Percent removals are also given using the values. Site IIA was not included in this analysis for two reasons: plant bypass rarely occurred at the site and, because of inaccurate plant flow measurements, the concentration (mg/l) of the pollutants in the influent and effluent could not be converted meaningfully to a mass basis.

The data collected in 1973 is not included here because it did not take into account the overflow volumes, if any, that occurred after the site run was ended. Therefore, the total overflow volumes for each run were not accurate. However, it should be pointed out that after the first flush is completed, the pollutional strength of the CSO decreases within two to three hours to a low, fairly constant value (see plot of discrete sampling events in Section VI, STORM WATER MANAGEMENT MODEL). In addition, screening/dissolved-air flotation treatment does not result in a significant improvement in the quality of the discharge; therefore, the benefits of treatment of the extended overflows are limited. For this discussion, the composite sample of the influent is used as the quality for the entire overflow event. Tables 40 and 41 give summaries of the percent total mass removals for Sites I and II, respectively.

The low percent removals achieved, especially at Site II, are due to the problems that occurred resulting in failure to capture large portions of the combined sewer overflow, especially the "first flush" overflow. Four recommendations are made to correct this problem:

1. The bar screen rakes at Site II should be kept operational at all times, if possible, so that a buildup of material on the bar screens does not block the plant flow and cause bypass.

2. The automatic startup mode for the sites should be kept operational and the sites should always be set to start automatically.

3. Equipment problems, when they occur, should be corrected while the sites are running, if possible. The sites should be shut down during an overflow only if absolutely necessary.

4. Personnel and chemicals should be available so the sites can be kept running until the combined sewer overflow has ended.

If these recommendations are followed, it will be possible to achieve a much greater percent removal of the pollutants discharging from the combined sewers.

Special Testing

Combined Sewer Overflow Pesticide Concentrations - The results of the pesticide testing are covered in Section V of this report, RIVER MONITORING STUDIES, and are considered in conjunction with the pesticide testing done on the Root River.

Combined Sewer Overflow Particle Size Distribution - Two separate sieve analyses were run on sewer samples composited from overflow discharge points No. 1 and 2 (see Figure 1) before site construction was completed. The analytical results for storm event No. 6 (8/10/71) are presented graphically in Figure 48, and the results for storm event No. 11 (11/1/71) are presented in Figure 49.

Sieve analyses were also made for the combined sewer overflow and final effluent at Site I. Figure 50 graphically presents the results of the sieve analyses performed on the combined sewer overflow and final effluent for run No. 12 (7/20/73). Figure 51 presents the results of the sieve analysis performed on the combined sewer overflow for run No. 42 (7/22/74).

TABLE 40. TOTAL MASS REMOVALS FROM COMBINED SEWER OVERFLOW VOLUMES - SITE I (1974)

Parameter	Mass in combined sewer overflow		Mass discharged to Root River		Percent removed
	kg	lb	kg	lb	
BOD	22,730	50,110	11,960	26,320	47.4
TOC	26,079	57,500	15,693	34,590	39.8
Total solids	183,999	405,600	146,667	323,400	20.3
Suspended solids	76,494	168,600	42,167	92,970	44.9
Volatile suspended solids	36,591	80,670	18,409	40,590	49.7
Phosphorus (as P)	823.0	1,814	532.1	1,173	35.3

TABLE 41. TOTAL MASS REMOVALS FROM COMBINED SEWER OVERFLOW VOLUMES - SITE II (1974)

Parameter	Mass in combined sewer overflow		Mass discharged to Root River		Percent removed
	kg	lb	kg	lb	
BOD	55,419	122,200	44,297	97,660	20.1
TOC	81,480	179,600	69,661	153,600	14.5
Total solids	743,324	1,639,000	670,397	1,478,000	9.8
Suspended solids	504,775	1,113,000	438,236	966,100	13.2
Volatile suspended solids	127,736	281,500	111,450	245,800	12.7
Phosphorus (as P)	1,619.8	3,571	1,398.2	3,082	13.7

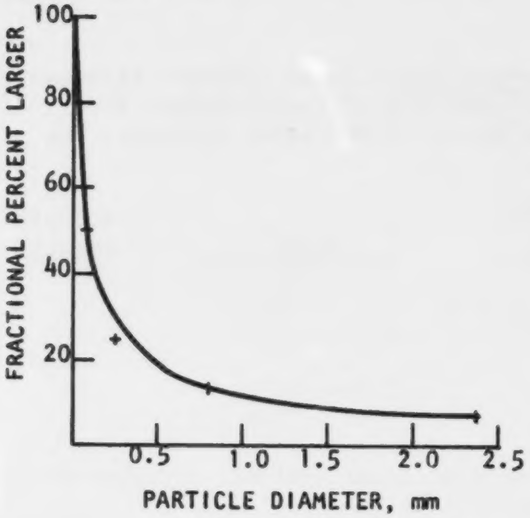


Figure 48. Sieve analysis of combined sewer overflow for storm No. 6, 1971.

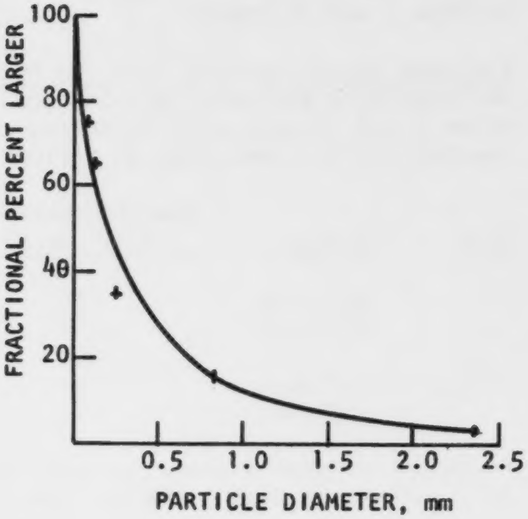


Figure 49. Sieve analysis of combined sewer overflow for storm No. 11, 1971.

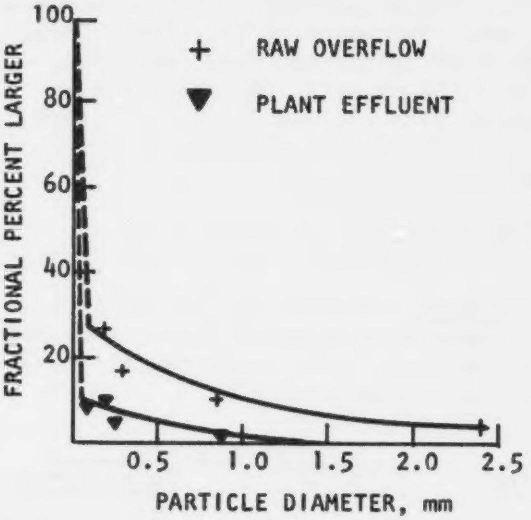


Figure 50. Sieve analyses of combined sewer overflow and plant effluent at Site 1, run No. 12, 1973.

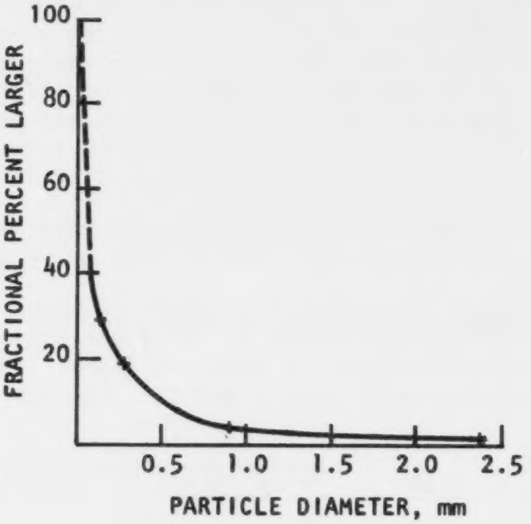


Figure 51. Sieve analysis of combined sewer overflow at Site 1, run No. 42, 1974.

The drum screens at Site I have an opening size of 0.297 mm. From the graphs presented, the average fraction of the particles greater than 0.297 mm was 22 percent. The average suspended solids removal by the drum screens at Site I was 32 percent.

Combined Sewer Overflow Nitrogen Concentrations - Total Kjeldahl nitrogen analyses were performed on both the influent and effluent samples from Sites I and II periodically during the course of the study project. The results may be summarized as follows:

Site	Sample	Mean concentration mg/l (as N)	Range	No. of events
I	Influent	8.58	2.55 - 13.20	6
	Effluent	5.52	2.95 - 7.40	6
II	Influent	4.50	2.30 - 6.70	2
	Effluent	2.10	1.60 - 2.90	3

These values give average percent removals of total Kjeldahl nitrogen of 35.7 percent at Site I and 53.3 percent at Site II.

Analyses were also performed for nitrogen and total dissolved solids in influent and effluent samples from both sites (I and II) for Run No. 12 (7/20/73). These analyses were performed after a long dry spell, 17 days since the last overflow event. The results are presented in Table 42.

Chloride Concentrations in Combined Sewer Overflows and Storm Sewer Discharges - Twice during the winter months, samples were taken of the combined sewer overflow at Site I and the storm sewer discharge at Site IIA. These samples were then analyzed for total chloride concentrations. The first set of samples was obtained on 12/26/73. The chloride concentration at Site I was 98 mg/l and the concentration at Site IIA was 795 mg/l. Since Site IIA receives mainly street runoff, this high concentration of chlorides is most likely caused by street salting operations.

The second set of samples was obtained on 3/28/74. The chloride concentration at Site I was 52 mg/l and the concentration at Site IIA was 58 mg/l.

Fecal Streptococci in Combined Sewer Overflows - For Run No. 12 (7/20/73), the concentrations of fecal streptococci were determined in addition to the concentrations of fecal coliforms in the combined sewer overflows and final effluents at Site I and II. The results follow (No./100 ml):

	Raw		Final Effluent	
Site	Fecal Coli	Fecal Strep	Fecal Coli	Fecal Strep
I	2,300,000	225,000	300	940
II	1,650,000	160,000	>10	970

TABLE 42. NITROGEN SERIES AND TOTAL DISSOLVED SOLIDS
FOR RUN NO. 12 AFTER LONG DRY SPELL
July 20, 1973 (17 Days Since Last Overflow)

Site No.	Parameter	Units	Raw	Final effluent
I	NH_3	mg/l - N	2.03	2.42
	NO_3	mg/l - N	1.01	2.30
	NO_2	mg/l - N	0.08	0.05
	TKN	mg/l - N	13.20	5.00
	TDS	mg/l	245	285
II	NH_3	mg/l - N	0.92	0.81
	NO_3	mg/l - N	1.34	1.08
	NO_2	mg/l - N	0.06	0.02
	TKN	mg/l - N	6.70	2.90
	TDS	mg/l	227	147

It appears that the destruction of fecal streptococci through the system is much less than the destruction of fecal coliforms.

IV-5 ECONOMIC CONSIDERATIONS

As stated previously, one of the objectives of this project is to develop detailed cost information (capital, operating, and maintenance costs) to establish cost/benefit relationship for this method of treatment compared to other treatment techniques. This information would also be helpful in evaluating the use of this treatment technique for other sites in Racine, as well as in other cities.

The design, land, construction, and equipment costs for the demonstration systems are presented in detail in Appendix IV-B. These costs were summarized previously in the subsection, IV-2, SYSTEM DESIGN AND CONSTRUCTION.

The capital cost for Site I was $436,599; Site II was $841,420; and Site IIA was $25,001. The total capital cost was $1,303,020 (March 28, 1973; ENR = 1973).

Using an interest rate of 8 percent, the following cost table (¢/cu m treated; ¢/1000 gal. treated) can be established for differing volumes treated per year and site life spans (amortization periods):

Volume treated Per year, cu m	Volume treated Per year, million gal.	¢/cu m (¢/1000 gal.) 20 yrs.	
378,500	(100)	35.1	(132.7)
757,000	(200)	17.5	(66.4)
1,135,000	(300)	11.7	(44.2)
1,514,000	(400)	8.8	(33.2)
1,691,000	(447)	8.0	(30.1)
1,892,500	(500)	7.0	(26.5)

The capital cost on a ¢/cu m treated basis decreases significantly as the volume treated increases during a year of operation.

The volume treated during the two years of the project was approximately 378,500 cu m (100 million gal.) per year. However, the plant bypass was 594,000 cu m (157 million gal.) per year. Treatment of the entire 972,500 cu m (257 million gal.) would have significantly reduced the capital costs on a ¢/cu m treated basis.

For the two years of the project, the sites averaged 23 runs per year, but these were not the total number of storm generated discharges. Assuming 40 discharge events per year, the total volume could be scaled up to 1,691,900 cu m (447 million gal.) (40/23 times 972,500 cu m (257 million gal.). If this estimated volume were all treated, the capital costs (amortized over 20 years) on a ¢/cu m treated basis could be reduced from 35.1 ¢/cu m (132.7¢/1000 gal.) to 8.0 ¢/cu m (30.1¢/1000 gal.).

Thirty and forty year amortization periods are presented because the sites are only run periodically during their life span. Therefore, it may be more accurate to estimate their life span to be longer than the standard 20 years used for sewage treatment plants which are run continuously. Using a longer amortization period, obviously, also reduces the capital costs on a ¢/cu m treated basis. For the 378,500 cu m (100 million gal.) treated per year during the project, 20 years yields a cost of 35.1¢/cu m (132.7¢/1000 gal.); 30 years gives 30.6¢/cu m (115.7¢/1000 gal.); and 40 years given 28.9¢/cu m (109.3¢/1000 gal.).

The operation and maintanance costs are presented in Table 43. The water cost is based on the yearly bills received. Electricity costs are the values (kwh) recorded from the electric meter during operation plus the amount used during nonoperational periods. Ferric chloride based on an addition rate of 40 mg/l. Polyelectrolyte costs are based on an addition rate of 2 mg/l of Nalco 607. Chlorine costs are based on an addition rate of 10 mg/l. Operation personnel costs are based on the presence of 2 men for an average of 4.85 hr for each discharge event. Included in the cost is the fact that 76 percent of their time will be outside of regular working hours

TABLE 43. OPERATION AND MAINTENANCE COSTS
August 1974; ENR = 2078

Item	Unit cost	¢/1000 gal.	¢/cu m
Water	$60.00/year	0.06	0.02
Electricity	$0.03/kwh	1.35	0.36
Ferric chloride	$5.52/100 lb	1.84	0.49
Polyelectrolyte	$0.41/lb	0.68	0.18
Chlorine	$10.25/100 lb	0.85	0.22
Operating personnel	$10.00/hr	3.08	0.81
Maintenance Personnel	$10.00/hr	8.58	2.27
Utility personnel	$9.00/hr	6.31	1.67
Miscellaneous (supplies, etc.)	$25/month	0.25	0.07
TOTAL		23.00	6.08
Chemicals (15% of total)		3.37	0.89
Utilities (6% of total)		1.41	0.37
Personnel (78% of total)		17.97	4.75

(40 hr/week) and the rate for this overtime was calculated as 1.5 times the base rate. A base rate of $10.00/hour is used to include the actual pay rate plus fringe benefits. This is based on an average wage rate for City of Racine Water Pollution Control Plant operating personnel. The maintenance cost is based on 30 man-hours/week. The total of 30 man-hours/week was estimated from the maintenance requirements during the last half of 1974 just before the project ended. The 30 man-hours/week were divided up into 13.5 man-hours/week for utility personnel who have a lower pay rate ($9.00/hour) and would be assigned to cleanup, and 16.4 man-hours/week for actual maintenance personnel. This breakup was made because it was found that 45 percent of available maintenance time was spent on cleanup activities (see Table 17).

Of the total operation and maintenance cost, 6.08¢/cu m (23.00¢/1000 gal.), 15 percent is for chemicals, 6 percent is for electricity and water, and 79 percent for labor. Of the labor cost, 4.75¢/cu m (17.97¢/1000 gal.), 17 percent is for actual operation, 48 percent for site maintenance, and 35 percent for site cleanup.

From experience gained in operating the sites, it is believed that operation and maintenance costs can be significantly reduced in one or more ways, to wit:

1. After the rainfall has ceased and the storm generated discharge has subsided, the sites could be left unattended to run until the discharge is over, especially for the long duration overflows at Site I. This approach could reduce the average time that personnel are present at the sites. The procedure could be that personnel be present at the sites for the first 2-1/2 hr of a discharge event and for 1/2 hr after it is over, thus resulting in an average of 6 man-hr for each event and both sites. Operating personnel cost reductions would be from 0.81¢/cu m (3.08¢/1000 gal.) to 0.50¢/cu m (1.91¢/1000 gal.).

2. If an automatic or semi-automatic method were available to clean the bottom of the flotation tanks, it is estimated that the cleanup cost could be cut in half (utility personnel), 1.67¢/cu m (6.31¢/1000 gal.) to 0.83¢/cu m (3.16¢/1000 gal.). Possible methods of cleaning are bottom scrapers or a steeply sloped tank with spray nozzles around the floor of the tank. These methods, of course, would increase the site construction cost.

3. If water, maintenance, and miscellaneous costs are assumed to be constant, their cost on a ¢/cu m treated basis can be significantly reduced if the volume treated is increased. As calculated previously, an estimated 1,691,900 cu m (447 million gal.) of discharge occurs at the sites yearly. If 1,514,000 cu m (400 million gal.) of the discharge were treated, the unit costs for water, maintenance, and miscellaneous would only be one-fourth as much as for 378,500 cu m (100 million gal.) treated.

If these procedures can all be achieved, estimated costs are presented in Table 44. The estimated total cost is 48 percent less than the cost incurred during the duration of the project (Table 43).

A comparison of the Racine actual costs and the Racine estimated costs to the costs reported by seven other sites treating storm generated discharges (16) is given in Table 45. The actual costs incurred at Racine are much higher than the other reported costs, and even the estimated operation and maintenance cost of 3.06¢/cu m (11.6¢/1000 gal.) is higher than the other reported costs.

The reason for the higher costs at Racine is the time required to maintain the sophisticated automatic controls and all of the required

TABLE 44. ESTIMATED FUTURE OPERATION AND MAINTENANCE COSTS
August, 1974; ENR = 2078

Item	Unit cost	¢/1000 gal	¢/cu m
Water	\$60.00/yr	0.02	0.01
Electricity	\$0.03/KWH	1.35	0.36
Ferric chloride	\$5.52/100 lb	1.84	0.49
Polyelectrolyte	\$0.41/lb	0.68	0.18
Chlorine	\$10.25/100 lb	0.85	0.22
Operating personnel	\$10.00/hr	1.91	0.50
Maintenance personnel	\$10.00/hr	2.15	0.57
Utility personnel	\$9.00/hr	3.16	0.83
Miscellaneous	\$25/month	0.06	0.02
TOTAL		12.02	3.18
Chemicals (28% of total)		3.37	0.89
Utilities (11% of total)		1.37	0.36
Personnel (60% of total)		7.22	1.91

TABLE 45. ACTUAL AND ESTIMATED OPERATION AND MAINTENANCE COSTS FOR RACINE COMPARED TO 7 COMBINED SEWER OVERFLOW TREATMENT SITES (ENR = 2000) (9)

Location	Type of process	Capacity		Operation and maintenance costs	
		cu m/day	mgd	¢/cu m	¢/1000 gal
Racine, WI (actual)	sdaf	219,530	58.0	5.85	22.1
Racine, WI (estimated)	sdaf	219,530	58.0	3.06	11.6
Fort Smith, AR	Hydraulic cyclone & daf	--	--	2.85	10.8
Milwaukee, WI	sdaf	18,925	5.0	1.53	5.8
Philadelphia, PA	Microstrainer	8,706	2.3	0.05	0.2
New Providence, NJ	Trickling filter	22,710	6.0	1.61	6.1
Kenosha, WI	Biological adsorption	87,055	23.0	1.27	4.8
Milwaukee, WI	Biological contactor	39,364	10.4	1.16	4.4
Springfield, IL	Oxidation pond	253,595	67.0	0.26	1.0

equipment. Table 45 also gives the design capacity of the sites reporting their operation and maintenance costs. Neglecting the Springfield oxidation pond, the Racine sites are more than double the size of any other CSO treatment site, and 10 times larger than the other dissolved-air flotation units. Therefore, for the small pilot units, maintenance costs are negligible, but for the full scale application of screening/dissolved-air flotation in Racine, it was found that maintenance costs will be a very large portion of the total costs of treatment.

SECTION V

ROOT RIVER MONITORING STUDIES

V-1 GENERAL DESCRIPTION OF THE ROOT RIVER DRAINAGE SYSTEM

In October 1965, the Southeastern Wisconsin Regional Planning Commission issued a comprehensive report titled: "Preliminary Report on a Comprehensive Development Plant for the Root River Watershed" (18). The following material is a condensation of the general descriptive information from that report and from a report issued by the State of Wisconsin Department of Natural Resources dated November 1967 (19).

Description of the Root River and Its Drainage Basin

The Root River rises near West Allis and flows south and east into Lake Michigan at Racine, a distance of 64 river kilometers (40 river miles). It drains about 500 square kilometers (193 square miles) of Kenosha, Milwaukee, Racine, and Waukesha counties. The watershed basin is bounded on the south by the Pike and Des Plaines watersheds and on the north by the Menomonee, Kinnickinnic, and Oak Creek watersheds. The western edge of the Root River Basin is bounded by the Fox River (Illinois) watershed; the western boundary also marks the Mississippi-St. Lawrence drainage basin divide (Figure 52).

Chief tributaries to the Root River are Hoods' Creek, which intersects the river about 19 kilometers (12 miles) from the mouth, and the Root River Canal, which flows into the main stem approximately 42 kilometers (26 miles) from the mouth. Figure 53 gives the locations of communities and industrial pollution sources found within the watershed.

In its 64 kilometers (40 miles) of length, the Root River falls nearly 46 meters (151 feet) resulting in an average gradient of about 0.72 meters per kilometer (3.8 feet per mile). In the final 9.7 kilometers (6 miles) of its length, however, the river exhibits a much steeper gradient, dropping about 3.0 meters per kilometer (15.9 feet per mile). The gradient of drop in the river and its tributaries is presented in Figure 54.

The topographical features of the Root River watershed are a result of glaciation. Although the area is composed chiefly of ground moraine, morainal hills and ridges are also encountered in the basin. The hills in the watershed reach an altitude of about 292 meters (960 feet) or approximately 116 meters (380 feet) above the level of Lake Michigan. Soils of the region are complex in pattern, although most can be classified as various types of silt loams. Extensive areas of poorly drained organic soils occupy

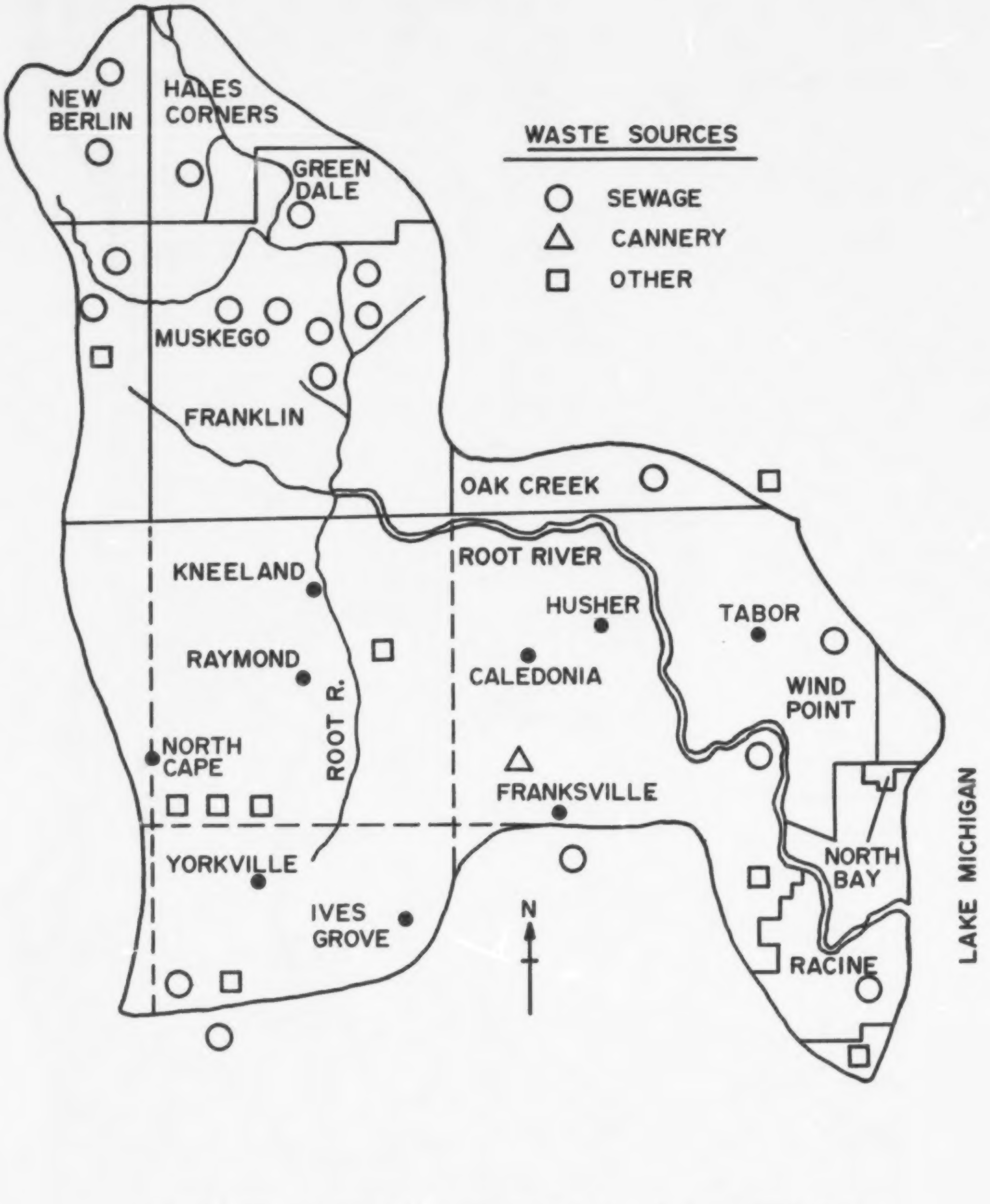


Figure 52. Root River watershed showing communities and principal industrial pollution sources.

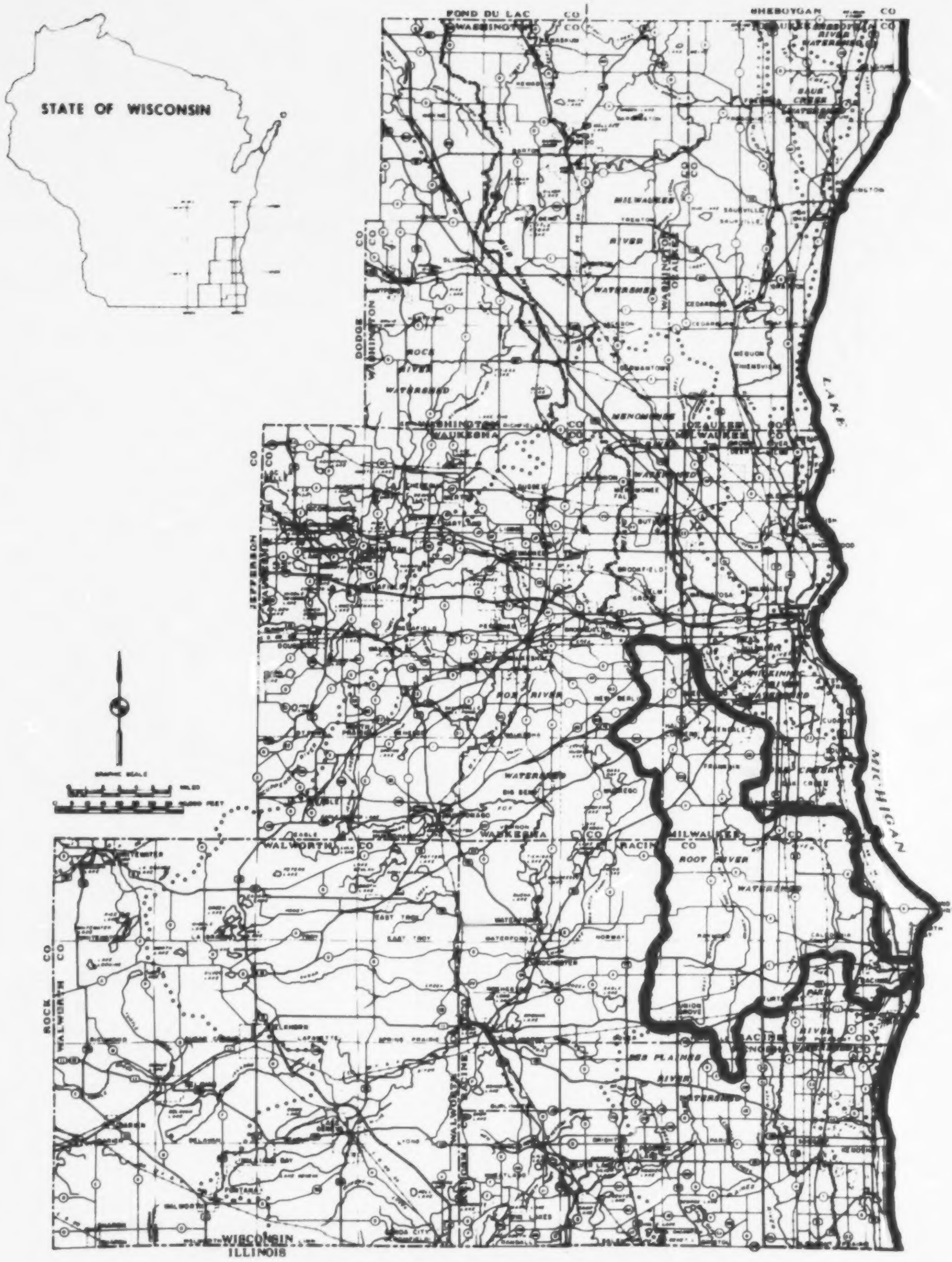


Figure 53. Location of the Root River watershed.

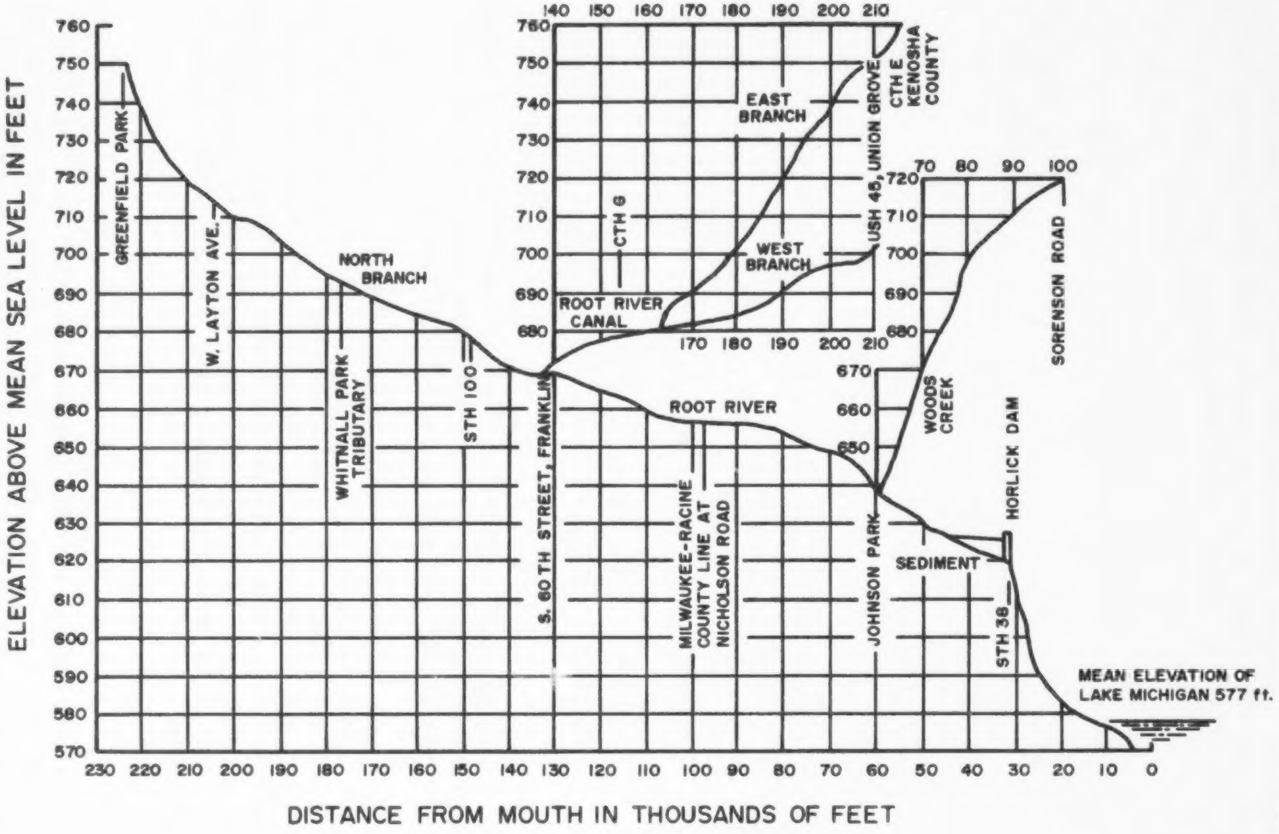


Figure 54. Profile of the Root River and its major tributaries (data base 1965, meters = feet x 0.3048).

certain areas of the basin. Virtually all of the soils encountered in the region have severe limitations for residential development utilizing septic tank disposal. Bedrock underlying the glacial drift consists of limestone, shale, and sandstone, with the drift, limestone, and sandstone being water bearing strata of chief importance.

Land Use

The distribution of land use by categories is roughly as follows:

Use category	Area				Percent of watershed area
	ha	ac	sq km	sq mi	
Residential	5,227	12,916	52.27	20.18	10.22
Commercial	236	583	2.36	0.91	0.46
Industrial	130	321	1.30	0.50	0.24
Mining	303	749	3.03	1.17	0.59
Transportation and utilities	3,828	9,459	38.28	14.78	7.50
Governmental and Institutional	471	1,164	4.71	1.82	0.92
Recreational	1,319	3,259	13.19	5.09	2.58
Agricultural	33,883	83,725	338.83	130.77	66.27
Water, woodland and wetland	5,739	14,181	57.39	22.17	11.22
	51,136	126,357	511.36	197.34	100.00

Urban land uses within the basin account for about one-fifth of the total area and are concentrated primarily within Milwaukee County on the upper tributaries of the Root River and within the City of Racine.

Approximately two-thirds of the land area in the basin is used for agricultural purposes. Some sections of the region are experiencing rapid urbanization. There are no natural inland lakes in the basin, and little recreational use is made of the streams. The relatively low streamflows frequently encountered, severely limit recreational, industrial, municipal, and agricultural uses of such waters. Lake Michigan is used extensively for recreational activities and as a water supply for the larger municipalities and industries.

Climate

The watershed has a continental climate characterized by four distinct seasons. Winter begins in November, lasts through March, and tends to be cloudy, cold, and snowy. Freeze-up of streams and lakes usually occurs in early December and does not end until early April; however, there is often a short-lived mid-winter thaw due to unseasonably warm temperatures. Spring is slow in arriving, partially because of the cooling effects of the waters of Lake Michigan, and is a mixture of both summer and winter.

Summers are fully developed and generally warm but marked by occasional hot and humid periods and unseasonably cool periods. Frequent breezes from Lake Michigan offer relief from high summer temperatures to those areas of the watershed lying within a few miles of Lake Michigan. Fall may extend from September to November and is characterized by mild, sunny days and cool nights. By Fall, Lake Michigan waters have become warm to the extent that the lake tends to prolong Fall in the watershed a week or so longer than in areas farther inland. The climate of the watershed can be understood more fully by examining phenomena of temperature, precipitation, wind movement, sunshine, and evaporation recorded at the Milwaukee First Order Weather Station, which is located within three miles of the watershed and is considered generally representative of watershed climatic conditions.

The mean daily temperature during the hottest month, July, is 21.84° C (71.35° F) with an official record high temperature of 38.30° C (101° F). The mean daily temperature during the coldest month, January, is -5.58° C (21.94° F) with an official record low of -31.08° C (-24° F). Temperature conditions within the watershed allow a growing season of from 155 to 175 days. Average dates of the last killing frost in spring and the first killing frost in fall are May 1 and October 12, respectively, with upland areas tending to have the most frost-free days.

Annual precipitation on the watershed, including snowfall, averages about 76 cm (30 inches), but annual amounts have ranged from a low of 47.58 cm (18.69 inches) to a high of 127.91 cm (50.36 inches). Most precipitation occurs as rain during the growing season (see Table 46). Most summer rainfall occurs in localized thunderstorms which usually move over the watershed in a few hours. However, 24-hour rainfall amounts of up to 19.05 cm (7.5 inches) (July 17-18, 1964) have fallen on the watershed as a result of a thunderstorm which became stationary over the watershed and was kept active by convergent winds.

Rainfall is often unevenly distributed during the growing season. Considering agricultural needs of about 2.54 cm (1 in.) of rainfall during each week of the growing season, the time distribution of rainfall within the watershed is relatively poor. The probability of this amount of rainfall occuring during each summer week ranges from a high of 4 in 10 years in early June and early August to 2 in 10 years in late July and late August.

Snow is the primary form of precipitation from late November through March. Although seasonal snowfall on the watershed averages about 102 cm (40 in.), individual seasons have ranged from 28 cm (11 in.) to 280 cm (110 in.). The probability of having snow on the ground reaches a high in mid-February and then decreases sharply. The actual water content of snowfall on the watershed varies with the individual storm, but averages about 10 percent, that is, 25.4 cm (10 in.) of snowfall is equivalent to 2.54 cm (1 in.) of rain.

Prevailing winds are westerly in Winter and southerly in the Summer over most of the basin; but within 0 to 5 km (0 to 3 mi) of Lake Michigan, northeasterly winds prevail during the period of April through June.

TABLE 46. MEAN MONTHLY PRECIPITATION AT MILWAUKEE, WISCONSIN (1854 - 1964)

Month	Mean cm	Precipitation inches	Percent of total
January	4.75	1.87	6.18
February	4.19	1.65	5.45
March	6.12	2.41	7.96
April	6.91	2.72	8.98
May	8.26	3.25	10.74
June	8.87	3.49	11.53
July	7.62	3.00	9.91
August	6.96	2.74	9.05
September	7.87	3.10	10.24
October	5.82	2.29	7.57
November	5.15	2.03	6.71
December	4.37	1.72	5.68
TOTAL	76.89	30.27	100.00

Wind speeds, neglecting gusts, can be expected to reach 88 km per hour (55 mi per hour) at the 9.1 m (30 ft) level and 72 km per hour (45 mi per hour) at the 0.3 m (10 ft) level in at least one out of two years. Speeds can be expected to reach 161 km per hour (100 mi per hour) at the 9.1 m (30 ft) level and 137 km per hour (85 mi per hour) at the 0.3 m (10 ft) level once in 50 years.

Sunshine over the basin occurs 55 percent of the maximum possible time during the year; 40 percent from November through February; 55 percent March through May and during October; 60 percent June through September and about 70 percent of the maximum possible during July.

Annual water surface evaporation is about equal to the mean annual precipitation of 76 cm (30 in.), but 80 percent of this demand on water supply occurs during the period May through October. Evapotranspiration from soils and plants is normally less than water surface evaporation, averaging about 53 cm (21 in.), most of which is demanded during the growing season. Depending upon such factors as land use, temperature, available water, and soil conditions, evapotranspiration will vary from 38 to 71 cm (15 to 28 in.).

Flow Characteristics

The quantity of streamflow varies widely from season to season and from year to year responding to variations in precipitation, temperature, soil moisture conditions, agricultural operations, the growth cycle of vegetation, and ground water levels. Since the quantity of streamflow is the product of many interrelated hydrologic factors, the only practical way to determine streamflow characteristics is to measure streamflow itself. In addition to the natural factors which affect streamflow mentioned above, it should be noted that effluent from several sewage treatment plants contributes to the flow of the Root River.

High streamflows occur principally in the late winter and early spring, usually associated with melting snow. Low flows persist for most of the remainder of the year with occasional rises caused by rainfall. Under present groundwater conditions, the lowest flows of the river appear to consist almost entirely of sewage disposal plant effluent, without which flows would probably drop to zero for considerable periods of time. In summary, river discharge generally responds much more to Winter and Spring rainfall than to Summer and Fall rainfall.

Water Quality Standards

The Root River has to meet the general use classification standards for intrastate waters as adopted by the Wisconsin Department of Natural Resources, September, 1973 (11). These include standards for fish and aquatic life and standards for recreational use.

Except for naturally occurring changes, the Root River shall meet the following criteria.

1. Dissolved Oxygen - Concentration should not be lowered to less than 5 mg/l at any time.

2. Temperature - There shall be no temperature changes that may adversely affect aquatic life. Natural daily and seasonal temperature fluctuations shall be maintained. The maximum temperature rise at the edge of the mixing zone shall not exceed 2.8° C (5°F). The temperature shall not exceed 31.6°C (89°F).

3. pH - The pH shall be within the range of 6.0 to 9.0 with no change greater than 0.5 units outside the estimated natural seasonal maximum and minimum.

4. Toxic Materials - Unauthorized concentrations of substances are not permitted that alone or in combination with other materials present are toxic to fish or other aquatic life. Questions concerning the permissible levels, or changes in the same, of a substance, or combination of substances of undefined toxicity to fish and other biota shall be resolved in accordance with the methods specified in "Water Quality Criteria", Report of the National Technical Advisory

Committee to the Secretary of the Interior, April 1, 1968 (20).

5. Recreational Use - The membrane filter fecal coliform count shall not exceed 2000 per 100 ml as geometric mean based on not less than five samples per month, nor exceed 400 per 100 ml in more than 10 percent of all samples during any month.

These standards are used as a basis for comparison in later sections where the water quality of the river is discussed.

Historical Water Quality of the Root River

Quality of water as conditioned by the natural environment of the watershed would present no problem for any reasonable possible uses of Root River systems waters. Most of the potential water uses have been, however, incompatible with past water quality factors resulting from human activity; principally disposal of waste and, to a lesser degree, agricultural and urban drainage.

A water quality sampling and testing program was carried out as part of the SEWRPC Regional Land Use - Transportation Study in 1964; a second study was done by the Wisconsin Department of Natural Resources during 1967 and 1968. Table 47 presents the data obtained from both of these studies. The findings of these investigations indicated that serious pollution problems existed in the Root River system and were intensifying.

As determined by the above studies, the variation of stream water quality with respect to location and season is extremely great. Figure 55 depicts the monthly variation of water quality parameters over the period 1961 to 1964 at the City of Racine. River discharge at the time of sampling has a strong influence upon the concentration of pollution factors. Since many pollutants are introduced into the River system at a relatively fixed flow rate, high streamflows result in a greater dilution than do low flows. From this historical data base it is easy to see that the natural stream purification potential is overwhelmed by the pollution load.

Root River Within the City of Racine

The test reach of the Root River, which for this project was defined as that portion of the river extending from Lake Michigan upstream to the area of the Horlick Dam (Figure 56), could be classified as a sluggish stream environment. This reach of the river covered a distance of 9.7 km (6.0 mi). Internal seasonal variance in Lake Michigan and artificial external environmental stress caused by heavy industrialization along its banks appear to keep the river in a state of chemical and biological flux.

The Root River has been dredged from the lakefront to State Street, a distance of nearly 1 kilometer (0.6 mi) in an attempt to encourage use of the waterway as a recreational focal point for private lake traffic. This abrupt drop in the normal river bottom gradient seems to act as a mixing

TABLE 47. WATER QUALITY OF THE ROOT RIVER AT HORLICK DAM
RACINE, WISCONSIN
1964-1968

Date of Sample	Total Coliform No./100 ml	BOD$_5$ mg/l	pH	Phosphorus (Total) mg/l	Solids (Total) mg/l	Solids (Dissolved) mg/l	DO mg/l	Temp. °C
1/30/64	32.0	5.1	7.2			640	9.6	0.6
2/19/64	0.4	4.3	7.2			955	11.5	0.0
3/25/64	0.7	3.5	8.5			750	14.6	4.4
4/3/64	21.0	5.7	7.1			600	12.3	5.0
5/25/64	10.0	4.7	7.4			770	9.0	20.0
6/11/64	1.5	2.7	8.2			800	7.9	20.6
7/16/64	6.0	4.0	7.8			635	8.1	25.0
8/6/64	3.0	8.1	7.7			620	7.5	24.4
9/11/64	26.0	6.3	8.2			665	8.1	19.4
10/2/64	17.0	5.7	7.8			720	9.3	15.0
11/5/64	1.0	5.5	8.2			750	10.5	11.1
12/3/64	16.0	3.8	7.6			620	12.0	0.0
1/6/65	20.0	5.1	7.4			505	12.3	1.1
2/3/65	2.0	4.8	7.2			735	7.3	0.0
1/15/65	17.0	5.3	7.7		941	940	12.4	0.0
2/25/65	4.1	3.0	7.5		462	447	10.4	0.5
3/31/65	52	5.7	7.8		380	294	12.4	2.0
4/27/65	--	4.7	7.6	0.52	530	468	11.4	8.5
5/24/65	160	8.9	8.0		568	484	7.1	15.0
6/28/65	.8	3.0	8.1		550	481	1.8	23.0
7/23/65	21	5.1	8.0		560	518	2.8	26.0
8/10/65	106	5.7	8.0	1.38	484	423	5.4	21.0
9/21/65	250	5.6	7.7		718	502	6.2	22.0
10/18/65	67.0	4.3	8.1		722	681	5.1	18.0
11/12/65	80.0	3.4	7.75		654	568	9.9	8.0
12/8/65	35.0	2.4	9.9		678	661	12.7	1.5
2/2/66	4.2	0.9	7.4	0.7	774	760	7.8	0.0
2/24/66	1.8	5.4	7.6		554	541	12.1	0.5
3/17/66	26.0	1.7	7.7	0.24	540	519	11.5	9.0
4/14/66	14.0	2.1	8.0		584	550	9.3	9.0

TABLE 47. (continued)

Date of Sample	Total Coliform No/100 ml	BOD_5 mg/l	pH	Phosphorus (Total) mg/l	Solids (Total) mg/l	Solids (Dissolved) mg/l	DO mg/l	Temp. °C
5/10/66	34.0	3.3	8.0		598	581	9.2	9.5
6/6/66	19.0	8.9	7.6		708	622	3.1	24.0
7/6/66	430	10.0	7.6	1.02	564	535	1.2	28.0
8/3/66	140	3.6	7.4		686	310	5.3	21.5
9/12/66	10.0	20.5	7.8	0.58	530	479	8.6	22.5
10/4/66	10.0	8.6	7.65		624	585	9.0	15.0
11/8/66	59.0	11.2	7.4		572	527	8.5	9.5
12/6/66	--	7.9	7.4		950	897	11.2	5.5
1/10/67	220	7.5	7.8	1.74	790	773	14.1	1.0
2/8/67	35.0	7.5	7.8		730	713	11.0	0.5
3/8/67	150	3.4	7.8		502	498	10.8	1.0
4/6/67	13.0	2.8	8.2	0.48	676	585	10.3	8.5
5/4/67	12.0	3.8	8.6		850	789	8.7	10.5
6/6/67	660	4.2	7.8	0.56	746	675	2.3	22.0
7/11/67	420	7.5	8.4		496	411	4.8	25.5
8/9/67	120	5.2	7.8		540	501	1.8	23.5
9/13/67	1.0	8.0	8.6	0.86	606	565	8.3	22.0
10/10/67	15.0	5.8	8.4		620	592	6.5	12.0
11/9/67	48.0	6.5	8.2		700	677	11.7	15.0
12/5/67	120	3.4	8.4		718	703	13.0	2.0
1/5/68	110	5.6	7.6	1.52	752	730	7.7	0.5
2/13/68	62.0	4.3	7.8		824	809	12.7	1.0
4/1/68	42.0	5.8	8.4	0.88	730	686	9.8	10.0
4/24/68	32.0	2.5	8.1		700	657	8.5	9.0
5/16/68	950	5.2	8.4		705	645	2.3	17.0
6/26/68	260	5.5	7.8		665	300	7.1	17.0
7/29/68	25.0	3.1	8.0		650	585		23.0
8/27/68	27.0	2.7	7.8	0.56	556	442	2.6	19.0
10/1/68	75.0	3.4	7.8		490	449	4.0	17.0
10/30/68	120	3.7	8.0		605	571	6.0	7.0
11/26/68	112	2.8	8.1	1.02	644	612	9.2	5.0
12/26/68	170	3.1	8.1		820	810	12.3	0.0

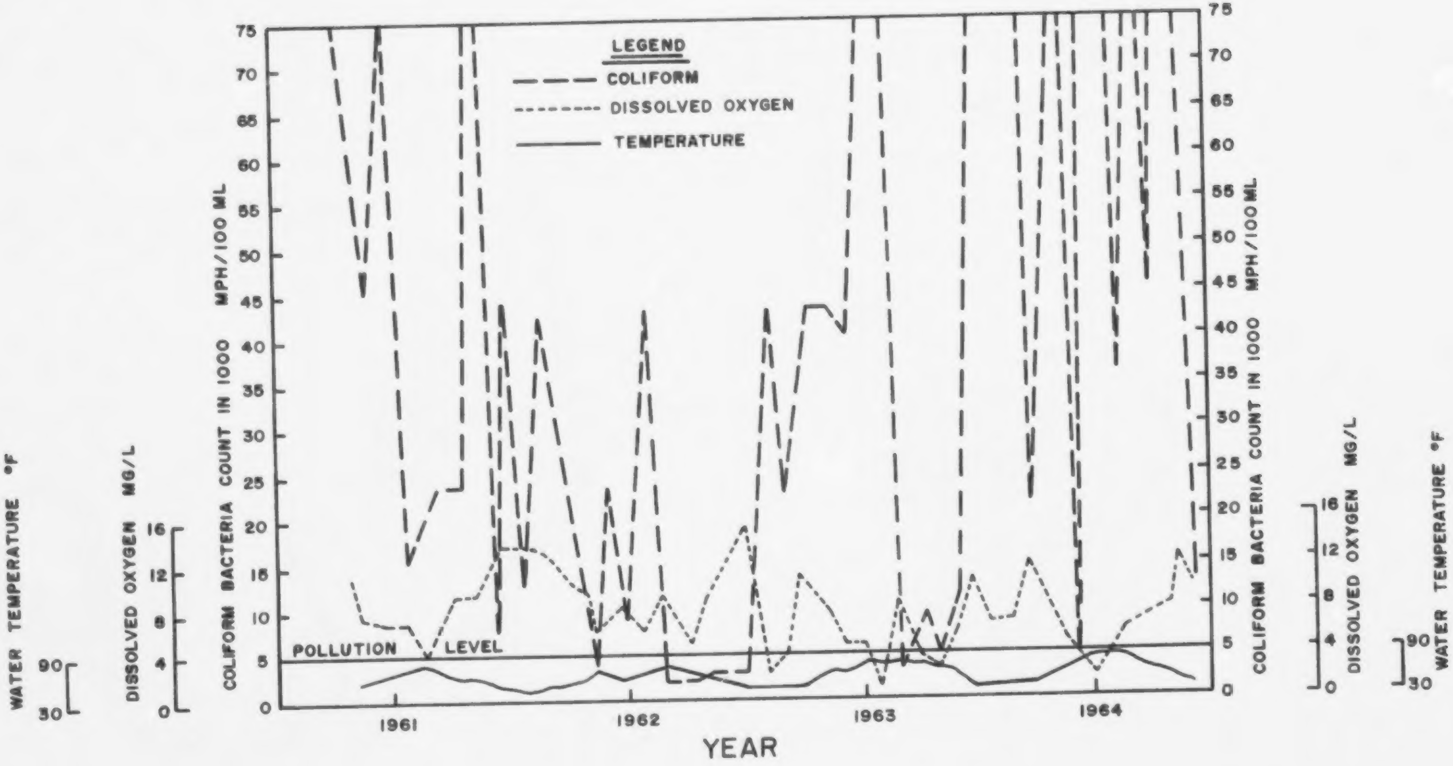


Figure 55. Water quality condition in the Root River at Horlick Dam (1961-1964).

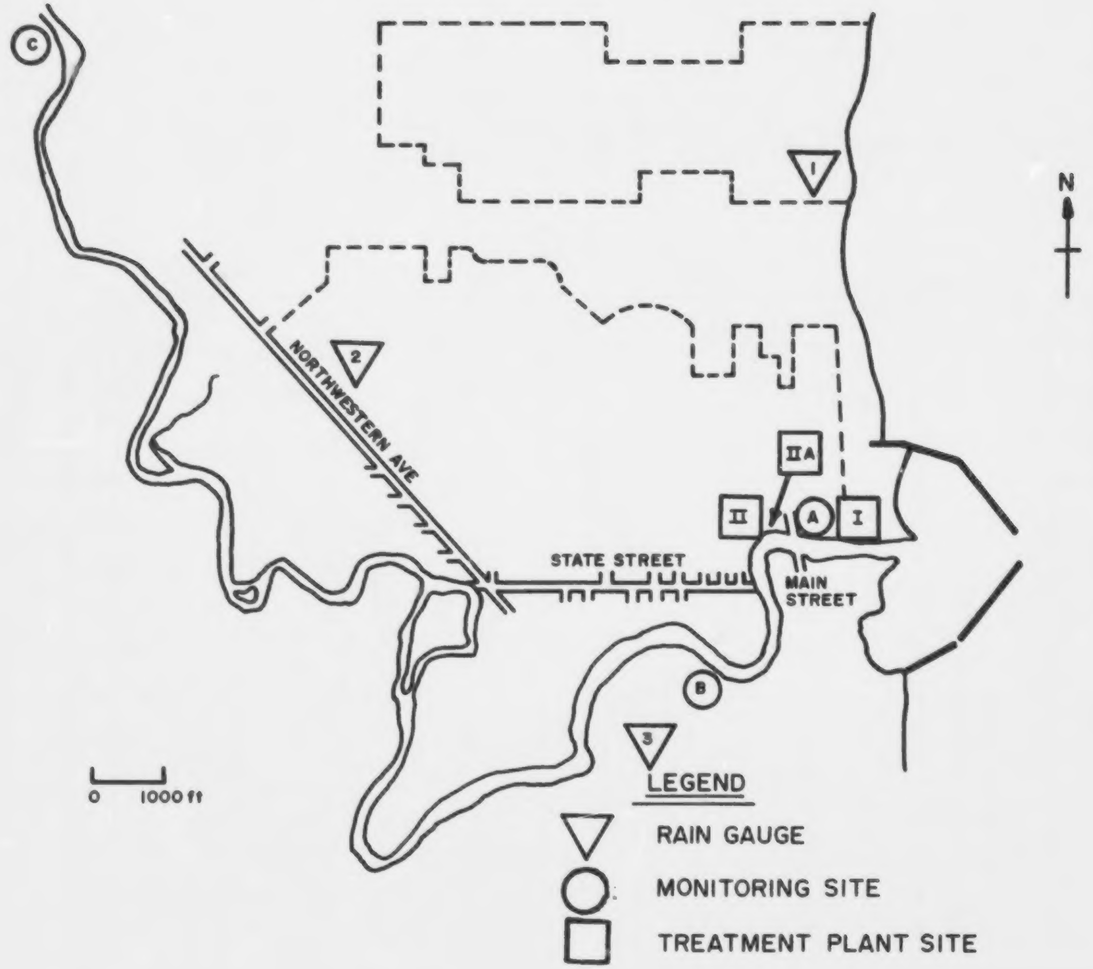


Figure 56. Root River test reach and monitoring points Point A, Point B and Point C.

zone between the waters of Lake Michigan and the river. (End of condensed material from Ref. 19.)

Root River Monitoring Sites

The purpose of the Root River monitoring program was to determine the effects of the treatment sites on the quality of the river in the test reach. To achieve this end, three monitoring sites were located at different points on the river.

One monitoring site was located at Horlick Dam, about 137 meters (150 yards) north of Wisconsin State Highway 38. This site, designated as Site C, was used as a control area and data gathered at this point indicated the water quality of the river just before it entered the City of Racine. In addition to acting as a source of water quality control data, this site was chosen because of its close proximity to a U.S.G.S. gauging station which gives a continuous readout of the stage of the river.

The second monitoring site, Site B, was located downstream of Site C, about 8.8 kilometers (5.5 miles) and 2.42 kilometers (1.50 miles) upstream of the river mouth. Site B was located on the southern bank of the river on the property of the Western Publishing Company. This site was selected as a monitoring point because it was, after the construction of the treatment sites, located downstream of the last known overflow points before the river enters the area covered by the treatment sites. Data obtained from this site indicated the water quality before the river entered the treatment site area and, through comparison of Site C with Site B, showed the effect of the upstream contribution to the river from the City of Racine.

The last site, Site A, was located in the test reach about 75 meters (246 feet) upstream of the outfall of treatment Site I and downstream, about the same distance, from the outfall of treatment Site II. Site IIA was located upstream of this monitoring site about 40 meters (131 feet).

Figure 56 depicts the relation of the monitoring sites on the river.

Additional Considerations for Monitoring Locations

Because of the nature of the monitoring program it was necessary to locate monitoring sites in areas which would be fairly concealed from the general public in order to avoid vandalism. Sites A and B were located using this as a consideration. Site C was located in an area where the general public had access, but because the area was frequented by many people, it was felt that vandalism, generally a private act, would be kept to a minimum.

The location of Site A upstream of one of the treatment sites was necessary because there were no sites between the Main Street Bridge and Lake Michigan, which would have allowed monitoring the river without interfering with the shipping channel.

V-2 METHODS USED TO MONITOR THE RIVER

Continuous, Analog Data Gathering

The continuous monitoring portion of the project utilized a Honeywell Water Quality Data Acquisition System at all three sites. The unit used at Site A was a Model 202F while the two remaining sites each utilized a Model I-W101. These monitoring systems were used to measure, in a continuous mode, dissolved oxygen, temperature, and conductivity, with the data being printed out on a strip chart. In addition to these parameters, wind velocity was also recorded at Site A.

Composite Sampling

This portion of the water quality monitoring studies consisted of taking discrete water samples once every hour from time zero of a storm (the time at which a storm caused an overflow) through hour No. 72. This sampling was done using Sigmamotor automatic samplers, Model No. WM-124R. Each sampler was capable of taking 24 discrete samples of 120 ml each. One of these samplers was placed at each monitoring site. When an overflow treatment event occurred, each sampler was turned on and the time recorded as time zero. Because each sampler only took 25 samples, the samples had to be collected every 24 hours and the samplers recycled. The 24 discrete samples were then composited into samples based on the time that they were taken. Samples taken on day one of a monitored period (0 to 24 hours) were composited into two 12-hour samples or into four 6-hour samples. Days two and three of each period had one 24-hour composite sample made up for each day. All composite samples were brought back to the laboratory and analyzed for:

- Solids
  - Total
  - Suspended
- Fecal coliform bacteria
- Total organic carbon
- Phosphorus
- Biochemical oxygen demand
- pH

Analyses were done according to the methods given in Appendix IV-D. A typical monitoring site with the equipment used for monitoring is depected in Figure 57.

Grab Sampling

There were occasions during the monitoring program that grab samples were taken from the river. This method of sampling was utilized when (1) samples were taken from places other than the three monitoring sites, or (2) during the early Spring prior to the placement of the sampling equipment at the respective monitoring sites.

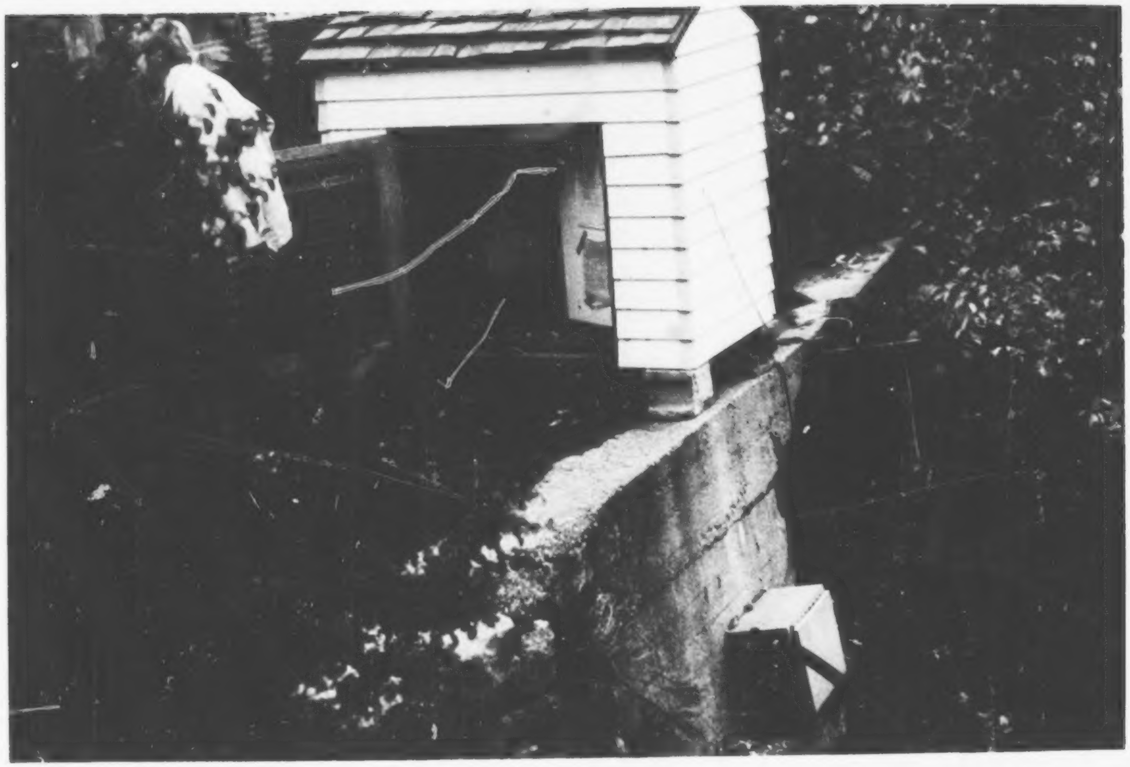


Figure 57. Root River monitoring station at Site C.

Miscellaneous Sampling

In addition to the methods described above, which cover a major portion of the Root River monitoring effort, various other techniques were used for additional studies designed to provide supplementary information on water quality. These techniques will be described when these studies are discussed.

V-3 RIVER MONITORING PERIODS

Determination of the water quality of the Root River was split into four different periods: 1971 dry weather studies, 1971 wet weather studies, 1973 wet weather studies, and 1974 wet weather studies.

Studies performed in 1971 provide information concerning the quality characteristics of the river prior to construction of the treatment systems. Data gathered during 1973 and 1974 give an indication of the quality of the Root River after the treatment systems went into operation.

All storm numbers assigned for the river monitoring periods during 1973 and 1974 correspond to the run numbers used in the discussion of the treatment sites.

1971 River Monitoring

The 1971 dry weather events consisted of monitoring periods performed after a period of at least two weeks had passed without a storm-generated discharge. Each event, with one exception, was monitored for three consecutive twenty-four hour periods. There were seven dry periods monitored during 1971. Table 48 is a list of these monitored events.

Twelve wet weather events were monitored during 1971. Table 49 is a list of the dates on which each of these events started. A wet weather event was designated as a rainfall which caused storm-generated discharge. During these events samples were taken while a discharge event was occurring at one-hour periods. Monitoring of each of these events ran for three consecutive 24-hr periods from the time that the first samples were taken.

1973 River Monitoring

During 1973, twenty-one storm-generated discharge events were monitored. Table 50 is a list of the starting dates of each of these events. Monitoring of these events was carried out in a pattern similar to that used during the 1971 storm events. Of the 22 events, only about 50 percent of these were monitored completely.

There were no dry weather periods monitored during 1973.

TABLE 48. MONITORED DRY WEATHER PERIODS-1971

Day	Month	Assigned monitor number	Monitored period number
9	June	1	1
10	June	2	
11	June	3	
30	June	4	2
1	July	5	
2	July	6	
14	July	7	3
15	July	8	
16	July	9	
Missed	(Data became part of a storm data)	10	
27	July	11	4
28	July	12	
29	July	13	
9	August	14	5
10	August	15	
7	September	16	6
8	September	17	
9	September	18	
10	November	19	7
11	November	20	
12	November	21	

TABLE 49. MONITORED STORM PERIODS - 1971

Day	Month	Assigned storm number
18	June	1
8	July	2
16	July	3
23	July	4
2	August	5
10	August	6
23	August	7
20	September	8
27	September	9
13	October	10
2	November	11
18	November	12

TABLE 50. MONITORED STORM PERIODS - 1973

Date Day	 Month	Assigned storm No.
29	April	1
1	May	2
7	May	3
8	May	4
25	May	5
27	May	6
5	June	7
15	June	8
16	June	9
2	August	10
3	August	11
20	August	12
4	September	13
17	September	14
21	September	15
24	September	16
28	September	17
12	October	18
27	October	19
31	October	20
14	November	21
4	December	22

TABLE 51. MONITORED STORM PERIODS - 1974

Date Day	 Month	Assigned storm No.
28	March	23
3	April	24
18	April	25
28	April	26
5	May	27
8	May	28
11	May	29
13	May	30
14	May	31
16	May	32
16	May	33
21	May	34
21	May	35
6	June	36
6	June	37
9	June	38
11	June	39
3	July	40
10	July	41
22	July	42
25	July	43
10	August	44
16	August	45

1974 River Monitoring

Twenty-three storm-generated discharge events occurred during 1974 which resulted in the operation of the treatment sites; these are listed in Table 51. Monitoring of storms during 1974 was 75% complete. Twenty-five percent of the data was missed due to equipment failure and/or because the monitoring equipment had not been installed during the early portion of the storm season.

Rainfall Measurement Program

Collection and characterization of rainfall events during the monitoring periods was important and necessary to overall project objectives. Data obtained from the U.S. Weather Bureau Station in Racine, the only station in the Root River Basin, indicated a mean annual precipitation of 82.45 cm (32.46 in.). The U.S. Weather Bureau Station in Milwaukee has recorded a 56 year mean precipitation of 75.39 cm (29.68 in.), ranging from a low of 47.7 cm (18.69 in.) to a high of 127.91 cm (50.36 in.).

Three raingages were installed at selected locations within the Racine area of the Root River drainage basin to provide detailed rainfall background data within the perimeter of the selected test reach. The raingage locations during the 1971 monitoring season are shown in Figure 56 and are designated as follows:

Gage R1 - on the roof of the Racine Police/Fire Headquarters Building at Center and 8th Streets;
Gage R2 - on the roof of the Racine Zoological Gardens Main Office Building one-half block east of Main Street on Walton;
Gage R3 - on the roof of Fire Station No. 2 at North Memorial Drive and High Street.

The raingages installed were all Bendix gages, Model No. 775-C. The Model 775-C has a knife edge collector of 20.3 cm (8 in.) inside diameter, constructed of nonferrous material. The catch is funneled into a bucket which has a 30.5 cm (12.0 in.) rainfall capacity. The bucket is mounted on a spring-type weighing mechanism which converts the weight of precipitation into its equivalent weight in inches of rainfall and actuates a pen which traces an inked record on a 15.2 cm (6.0 in.) revolving chart. The record is of the dual traverse type; the pen sweeping across the chart from bottom to top for 15.2 cm (6.0 in.) of rainfall and then reversing to return to the bottom for an additional 15.2 cm, thus recording 2.54 cm (1.0 in) of rainfall for each 2.54 cm (1.0 in.) of chart. The chart is attached to a vertical cylinder which is rotated by an internal drive spring once per day for an eight day period. The accuracy of the 775-C gage is 1/2 of 1 percent of full scale (± 0.15 cm) (0.06 in.).

Each site was maintained on a seven-day interval or after each rainfall, depending on which came first. The maintenance included changing the recording chart, refilling the recording pen, and checking the instrument calibration. Summaries of the collected rainfall data during the 1971, 1973, and 1974 monitoring periods are included in Tables 52, 53, and 54, respectively.

The notification and alarm system which initiated a wet weather sampling period was very closely tied to the rainfall measurement program. Selected

TABLE 52. RAINFALL SUMMARY FOR RACINE, WISCONSIN FOR 1971

Date rainfall event	Total rainfall cm	Total rainfall (inches)	Duration hours	Average intensity cm/hr	Average intensity (inches/hour)
6/18 M[a]	1.14	(0.45)	0.50	2.29	(0.90)
6/20	0.25	(0.10)	0.66	0.39	(0.15)
7/2	1.78	(0.70)	1.16	0.53	(0.60)
7/8 M	2.67	(1.05)	3.66	0.73	(0.29)
7/16 M	1.07	(0.42)	0.66	1.62	(0.63)
7/19	1.35	(0.52)	1.33	1.01	(0.39)
7/20	0.25	(0.10)	0.16	1.59	(0.60)
7/23 M	0.86	(0.34)	1.16	0.74	(0.29)
8/2 M	0.74	(0.29)	2.83	0.26	(0.10)
8/10 M	2.16	(0.85)	3.00	0.72	(0.28)
8/18	1.52	(0.60)	2.50	0.61	(0.24)
8/22 M	1.32	(0.52)	0.50	2.64	(1.04)
9/5	0.89	(0.35)	6.00	0.15	(0.06)
9/20 M	1.73	(0.68)	4.33	0.40	(0.16)
9/29	1.02	(0.40)	0.16	6.35	(2.40)
9/29	0.46	(0.18)	0.33	1.39	(0.54)
10/3	2.67	(1.05)	1.00	2.67	(1.05)
10/13	0.56	(0.22)	2.66	0.21	(0.08)
10/21	0.69	(0.27)	18.00	0.04	(0.62)
11/1 M	0.99	(0.39)	1.33	0.75	(0.29)
11/18 M	0.31	(0.12)	1.33	0.23	(0.09)

a. Indicates a monitored storm event.

TABLE 53. RAINFALL SUMMARY FOR RACINE, WISCONSIN FOR 1973

Date of rainfall event	Total rainfall		Duration hours	Average intensity		Run No.
	cm	(inches)		cm/hr	in/hr	
4/27	1.78	(0.70)	5.25	0.34	(0.13)	1
5/1	1.78	(0.70)	2.00	0.89	(0.35)	2
5/7	-	-	-	-	-	3
5/8	-	-	-	-	-	4
5/25	0.97	(0.38)	1.78	0.54	(0.21)	5
5/27	2.54	(1.00)	15.00	0.17	(0.07)	6
6/5	0.99	(0.39)	0.58	1.71	(0.67)	7
6/15	0.48	(0.19)	1.31	0.37	(0.15)	8
6/16	0.58	(0.23)	0.50	1.17	(0.46)	9
7/2	-	-	-	-	-	10
7/3	-	-	-	-	-	11
8/20	4.65	(1.83)	3.00	1.55	(0.61)	12
9/4	3.73	(1.47)	1.72	2.17	(0.85)	13
9/17	2.57	(1.01)	13.90	0.18	(0.07)	14
9/21	0.58	(0.23)	2.00	0.29	(0.12)	15
9/24	1.52	(0.60)	4.67	0.33	(0.13)	16
9/28	3.68	(1.45)	7.58	0.49	(0.19)	17
10/12	1.78	(0.10)	10.42	0.17	(0.07)	18
10/27	1.58	(0.62)	14.53	0.11	(0.04)	19
10/31	1.45	(0.57)	9.64	0.15	(0.06)	20
11/14	2.36	(0.93)	7.92	0.30	(0.12)	21
12/4	3.25	(1.28)	14.36	0.23	(0.09)	22

TABLE 54. RAINFALL SUMMARY FOR RACINE, WISCONSIN FOR 1974

Date of Rainfall event	Total Rainfall		Duration, hours	Average Intensity		Run No.
	cm	inches		cm/hr	in/hr	
3/28	+ --	--	--	--	--	23
4/3	1.57	0.62	4.8	0.32	0.13	24
4/18	0.66	0.26	3.3	0.20	0.08	25
4/28	3.81	1.50	4.5	0.85	0.33	21
5/5	0.30	0.12	1.5	0.20	0.08	27
5/8	1.09	0.42	11.2	0.10	0.04	28
5/11	1.14	0.45	7.5	0.15	0.06	29
5/13	0.51	0.20	0.8	0.11	0.24	30
5/14	1.32	0.52	4.0	0.33	0.13	31
5/16	1.98	0.78	5.0	0.40	0.16	32
5/16	0.53	0.21	3.8	0.14	0.06	33
5/21	0.64	0.25	2.2	0.30	0.12	34
5/21	1.68	0.66	14.0	0.12	0.05	35
6/6	0.53	0.21	4.5	0.12	0.05	36
6/6	1.52	0.10	5.3	0.28	0.11	37
6/9	2.59	1.02	5.0	0.52	0.20	38
6/11	0.41	0.16	1.0	0.41	0.16	39
7/3			13.0			40
7/10	1.22	0.48	9.7	0.13	0.41	41
7/22	0.78	0.31	1.0	0.79	0.31	42
7/25	0.33	0.13	2.8	0.12	0.05	43
8/10	0.51	0.20	2.5	0.20	0.08	44
8/16	1.42	0.56	6.0	0.24	0.09	45

personnel from the Racine Water Pollution Control Plant were delegated the responsibility of notifying the appropriate Envirex personnel in the event of a rainfall occurrence during 1971. On notification, the designated Envirex employee immediately proceeded to Racine to ensure against equipment malfunction and to collect the initial samples for that period. No alarm system was necessary for the monitoring of dry weather flow. During 1973 and 1974 an automatic telemetered notification system was used to inform Envirex employees that an overflow event was occurring.

The U.S. Geological Survey maintains a gauging station at the head end of the test reach, about 200 meters (650 feet) downstream of Site C river monitoring station. The Madison, Wisconsin office of the Geological Survey made up-to-date discharge data for this station available for use. An estimate of the stability of discharge at this site during 1971, 1973 and 1974 wet weather monitoring periods was obtained by computing the mean monthly discharge from the daily discharge measurements and calculating the coefficient of variation (21) of the daily measurements around the monthly mean. The mean monthly discharge and coefficients of variation for discharge are shown in Table 55. During the 1971 monitoring period the river discharge was extremely low when compared to 1973 and 1974 data. Discharge measurements during 1973 and 1974 were similar with discharge measurements during 1974 being slightly higher than those in 1973.

Specific Conductance

All natural waters contain ionized materials and the amount of this material in a given system can be estimated by measurement of its specific conductance or conductivity. Units of measure for specific conductance are expressed in terms of micromhos per centimeter at 25°C. It is the reciprocal of the specific resistance of a solution. In most bodies of natural waters the relationship of specific conductance to dissolved solids is linear (22). Because of this relationship, either parameter may be used as an approximation of the other. This relationship was confirmed in the Root River water samples by running determinations of each parameter in the laboratory and using the data generated in a statistical analysis. The analyses used consisted of using "six curves", "best fit" and "regression analysis" programs, which are part of a statistical package offered by the Service Bureau Corp. computer center (23). The output from these analyses is presented in Figure 58. These analyses showed that a linear relationship existed between the two parameters with the correlation coefficient having a value of 0.867.

Measurement of specific conductance was done through the use of three Honeywell Water Quality Data Acquisition Systems, one at each of the three major sampling sites. Checking the readout of these instruments with external instrumentation (Beckman Instrument Co., Model No. RB-3) indicated that measurements of this parameter by the Honeywell units were very accurate (± 5%).

Specific conductance measurements were made during 1971, 1973, and 1974 with data from 1971 being split into two periods: 1971 wet weather, and 1971 dry weather. All data for 1973 and 1974 were taken during wet weather

TABLE 55. MONTHLY MEANS AND COEFFICIENTS OF VARIATION FOR DISCHARGE OF THE ROOT RIVER AT RACINE FOR MONTHS MONITORED

	April	May	June	July	Aug.	Sept.	Oct.	Nov.
1971								
Mean Discharge (cu m/min)			54	32	11	9	16	23
Mean Discharge (cfs)			32	19	7	5	9	13
Coefficient of Variation			0.5306	1.2599	0.6093	0.7681	0.7423	1.0591
1973								
Mean Discharge (cu m/min)	1329	624	264	60	23	39		
Mean Discharge (cfs)	782	367	155	36	14	23		
Coefficient of Variation	0.9675	0.8181	0.7030	0.4534	0.2792	1.2271		
1974								
Mean Discharge (cu m/min)		804	352	36	28			
Mean Discharge (cfs)		473	207	21	17			
Coefficient of Variation		0.8592	1.1762	0.2933	0.5005			

	Curve Type	Equation of Best Fit	Correlation Coefficient
1.	Linear	Dep = -4.8400 + 1.25931 x Ind	0.86748
2.	Exponential	Dep = 193.53408 Exp(0.002.5 x Ind)	0.82608
3.	Power Function	Dep = 1.47821 x Ind Exp 0.9689519	0.8636
4.	Hyperbolic	Dep = 1058.10233 - 1/Ind	0.74427
5.	Hyperbolic	Dep = 1/(0.00403 - Ind)	****
6.	Hyperbolic	Dep = Ind/(0.66117 + 0.00040 x Ind)	0.79771

Figure 58. Computer printout showing relationship of dissolved solids concentration (dependent variable) to specific conductance (independent variable)

periods. Measurements of specific conductance for a wet weather period were run for three consecutive days (day 1, day 2, and day 3) with day 1 being the day the rainfall and resultant discharge occurred. During the 1971 dry weather studies only six of the seven three day periods were monitored for this parameter. Wet weather monitoring during 1971 occurred during all twelve storm events. During 1973 a total of 22 storm-generated discharge events occurred of which nineteen were monitored for this parameter. In 1974, a total of 23 events occurred of which 21 were monitored for this parameter. Statistical analysis of the data, at the 95% confidence level, was carried out comparing years and sites.

In all four of the monitoring periods there were no significant differences noted at Point C although there was a decrease in this parameter during 1974 compared to the other monitoring periods. Analysis of data at the other two points did show a significant increase in this parameter between 1971 and 1973 for wet weather data, as shown in Table 56, as well as a significant difference between 1971 wet and dry weather studies. These analyses indicate that the water quality in the river has decreased between 1971 and 1973, using specific conductance following a discharge event as the indicating parameter. Comparison of specific conductance values between 1973 and 1974 at Point A indicates that, as happened from 1971 to 1973, the quality of the River decreased significantly. This same trend was noted at Point B It can be inferred from the data that degradation of Root River water quality is being caused by some input upstream of the Horlick Dam.

Temperature

Rivers and streams, as opposed to lakes, have a temperature regime that fluctuates daily, as well as seasonally. The amount of fluctuation observed in a running body of water depends a great deal on the depth, width, and volume of flow. In general, the deeper the body of water the less the variation will be in its daily temperature cycles, as well as in its annual temperature cycle. The temperature of a river or stream is related to the air temperature of the area that it passes through. Due to the fact that rivers and streams are moving bodies of water, there is little or no thermal stratification over the course of the year. If a period of stratification does occur, it only lasts for a very short interval because of the constant mixing action of the moving water.

Temperature affects the biota in a body of water directly and indirectly. The direct effect is related to the fact that each organism has a temperature range that it can tolerate. The indirect effect is related to the amount of dissolved oxygen (DO) that is soluble in water. The relationship of DO to temperature is an inverse one, that is, as the temperature increases, the solubility of oxygen in water decreases.

For the stretch of the Root River being studied, the greatest daily variation in temperature is found at Point C. The physical makeup of the river at that point is a shallow water area that is not shaded by trees and therefore receives all of the sun's solation. Point A has a lower value at the high end of its range, but this is probably due to the influence of

TABLE 56. SPECIFIC CONDUCTANCE GRAND MEANS FOR ALL SAMPLING PERIODS (μ mhos/cm)

Sampling Period	Main Street		
	Day 1	Day 2	Day 3
Dry 1971	217	226	222
Storm 1971	245	252	258
Storm 1973	422	420	425
Storm 1974	514	528	526

Sampling Period	Western Publishing		
	Day 1	Day 2	Day 3
Dry 1971	414	389	369
Storm 1971	504	532	575
Storm 1973	760	723	694
Storm 1974	624	614	620

Sampling Period	Horlick Dam		
	Day 1	Day 2	Day 3
Dry 1971	820	819	856
Storm 1971	803	784	774
Storm 1973	838	782	781
Storm 1974	734	743	720

Lake Michigan at this site. Site B is similar to Site A except the "lake effect" is not as pronounced.

The effect of storm-generated discharges on the temperature regime of a river system is dependent on two factors: first, the season of the year which in turn affects the temperature of precipitation received; second, the temperature of the surface that the runoff water travels across which is also dependent on the reason of the year. These two characteristics of storm events and their effects on runoff determine the final temperature of runoff water as it enters a receiving body. The added water, with its temperature characteristic, will change the temperature of the receiving body of water. Appendix V-A, Tables A5, A6, A7, and A8 give the daily mean temperature and daily temperature maximum and minimum of the river at each of the three sites for each day of the monitored period.

Dissolved Oxygen

The amount of dissolved oxygen in a moving body of water is highly dependent on many different factors. These include size of the body of water being examined, amount and type of pollutional load it carries, water temperature and the biota that live in its waters. It has also been demonstrated in large rivers that one of the main influences on the concentration of dissolved oxygen is the flow rate of the river.

In rivers (and other bodies of water) which receive inputs of organic matter, such as the inputs from storm-generated discharges, the amount of oxygen needed on a per day basis is increased above the amount needed to maintain the indigenous biotic community. If the input of organic matter (as well as other oxygen requiring pollutants) from point sources is great enough, it may reduce the oxygen concentration in the water below the lower tolerance of the respiratory needs of the biota, placing them in a stressed situation.

Data for each of the days monitored is expressed as a mean for that day along with the minimum and maximum values recorded for that day. Tables A9, A10, A11, and A12 of Appendix V-A give this information for the four major sampling periods. The grand mean, minimum and maximum values, in mg/l, recorded over the three major sampling periods at each point are as follows:

Dissolved oxygen content (mg/l)

	1971 Dry weather	1971 Wet weather	1973 Wet weather	1974 Wet weather
Point A				
Mean	8.1	7.1	7.2	8.3
Minimum	5.2	3.7	0.8	5.0
Maximum	10.8	10.4	11.8	11.8
Point B				
Mean	5.4	5.8	6.1	7.4
Minimum	1.0	1.7	1.0	0.0
Maximum	8.5	10.5	13.6	16.4
Point C				
Mean	2.3	2.8	6.9	7.4
Minimum	0.1	0.1	0.1	0.0
Maximum	5.5	9.0	11.4	17.1

Analysis of the data indicated that there was a decrease in the mean dissolved oxygen value at Point A from dry to wet weather for 1971. At the same time, an increase of this parameter for the same period of comparison was noted at the other two points. Also, there were no significant differences between the sampling points when the means for all monitored events were compared; this comparison included all 1971 and 1973 data.

Comparison of 1971 storm data indicates that, when all points are compared, a significant difference does exist between points. The trend shows that a general improvement in water quality occurs from Point C downstream to Points A and B. When the 1971 and 1973 storm data are compared by point, an overall improvement in water quality occurred from 1971 to 1973 along the entire length of the test reach. The greatest improvement in water quality during the entire monitoring period occurred at Point C where the difference between 1971 and 1973 storm events proved to be statistically significant. Statistical comparison of 1971 dry and wet weather events at this site show no significant differences. Analysis of data gathered at Point B revealed no significant differences between 1971 and 1973 storm data or 1971 wet and dry weather data. Dry and wet weather events of 1971 were compared for Point A. Statistical analysis revealed that the difference between the two periods at this Point was significant with a decrease in this parameter being noted from dry to wet weather events.

The data gathered in 1974 showed an increase in this parameter at all points when compared with the three previous monitoring periods. Statistical comparison of 1973 data to 1974 data at Point A indicated that the differences during day-1 and day-2 of the monitored events was significant while the differences noted during day-3 were not significant. These differences indicated an improvement in water quality from 1973 to 1974. Comparison of 1973 data to 1974 data for each of the days monitored at the two remaining monitoring points indicated no significant differences, although the general trend was an overall improvement in water quality. Comparison of all points to each other during 1974 indicated that no significant differences existed.

Solids

All natural waters contain a certain amount of solids. These solids may be divided into two components, dissolved and suspended. In this study the difference between these two fractions is based on the ability of dissolved solids to pass through a glass fiber filter while the suspended solids are retained. Collectively, these two components make up the total solids load of a body of water. The material which makes up the solids load is both organic and inorganic although it has been stated that the true solids component is only made up of inorganic material (24). Historically, the dissolved solids content of a water body has been related to its fertility as well as to its specific conductance. The relationship of dissolved solids to conductivity on the Root River has been discussed previously in this section.

The input of solids to many streams and lakes in the United States is derived, in many cases, from the waste discharged by industry and municipal waste treatment plants. Effluents from these sources are generally higher in dissolved solids than they are in suspended solids. A major portion of the suspended solids load results from soil erosion and/or algae blooms. The effect of a high suspended solids load on a water body is to inhibit primary production by decreasing the amount of light entering the system and/or changing its quality.

During the four monitoring periods (1971 Dry Weather, 1971 Wet Weather, 1973 Wet Weather, and 1974 Wet Weather) measurement of total solids and suspended solids were made. Dissolved solids content was calculated from the difference between the two. Appendix V-A contains the values obtained by these measurements. Statistical analysis of these results was accomplished through the use of the Student t Test or an analysis of variance test. All statistical analyses were carried out at the 95% confidence level.

Total Solids - During the four major monitoring periods, determinations were made of the total solids content of the Root River. Tables A13 to A16 in Appendix V-A give the values obtained by these measurements for all of the monitored periods.

Statistical comparison of data for this parameter during the two separate monitoring periods of 1971 (wet and dry weather) at each of the monitoring points indicates that there were no significant differences between wet and dry weather flow. When points were compared to each other for each of the two 1971 monitoring periods, significant differences were observed, with an increase in the parameter proceeding upstream from Point A to Point C (see Table 57) during both dry and wet weather.

Wet weather data comparisons between 1971 and 1973 at each site indicate that there was no significant change at Point C, but that significant changes did occur at Points A and B. The change in this parameter was a 37% and 27% increase for Point A and Point B, respectively, from 1971 to 1973.

Comparison of all points during 1973 again showed that the differences encountered were not significant. The differences between Point A and the other two points were significant during 1973. Again, there was noted an increase in the parameter from Point A to Point C, although the increase was not as great as it was in 1971.

Data on total solids for 1974 showed that there had been an increase in the parameter from 1973 at Point A although the increase was not statistically significant. When the 1974 data was compared at Points B and C, no significant differences were noted.

When the first six hours of the 1974 wet weather events were compared across points the differences noted were not significant but the next three six-hour periods have significant differences between points.

Through use of the Student t Test, it was found that, during 1974, Points B and C were similar but that they both were statistically different from

TABLE 57. TOTAL SOLIDS - MEAN VALUES FOR ALL MONITORED PERIODS (mg/l)

	Hours							
	0-6	7-12	13-18	19-24	0-24	25-48	49-72	Site
1971 Dry	--	--	--	--	218	241	213	A
1971 Wet	230	219	205	238	225	244	270	A
1973 Wet	389	402	361	398	385	387	396	A
1974 Wet	518	464	461	477	465	457	433	A
1971 Dry	--	--	--	--	416	435	383	B
1971 Wet	384	406	424	417	407	445	453	B
1973 Wet	626	615	586	620	613	558	622	B
1974 Wet	614	623	576	644	617	622	612	B
1971 Dry	--	--	--	--	730	726	741	C
1971 Wet	647	653	754	687	687	715	705	C
1973 Wet	674	661	672	641	662	672	618	C
1974 Wet	651	681	673	697	680	673	694	C

Point A, Point A having a lower mean value for this parameter than the other two points.

When comparisons of 1971 wet and dry weather data are made, there is a noticeable amount of fluctuation during the wet weather periods which is not apparent during dry weather periods. This trend is most predominant at Points A and B which is to be expected because of the input from the city streets via storm sewers and combined sewer overflows during runoff events, as well as the influence from the lake.

Suspended Solids - Tables A17-A20 (Appendix V-A) present data on the amount of suspended solids found in the river. Point C had the highest amount of suspended solids and Point A had the lowest amount during periods of low flow (1971 dry weather). Comparison of 1971 wet and dry weather flow at Point C and Point B showed no significant differences when compared using the Student t statistic. There was a significant difference in this parameter indicated at Point A. Comparison of the two different wet-weather monitoring periods at each site reveal that suspended solids were not significantly different at Point C between 1971 and 1973, but that there was a significant difference at the other two points. The 1973 values at Point B and Point A were greater than the 1971 values; the 1973 values were

94% and 113% greater, respectively, than the 1971 values. The increases are probably a result of events which are occurring somewhere along the reach of the river in Racine, such as outfalls from combined sewers and storm sewers, as well as an overall increase in river discharge.

Events monitored during 1974 showed a significant increase in the parameter at Point A from 1973. Suspended solids during 1974 were 150% higher than they were during 1973 at this point. Although there were increases noted at Point B and C from 1973 and 1974, the increases were not statistically significant. The percent increase at these two points from 1973 to 1974 were 100% and 169% for Points B and C, respectively.

Dissolved Solids - Dissolved solids, as indicated by the data in Tables A21-A24 in Appendix V-A, make up a major portion of the total solids content in the test reach of the Root River, and reflect similar trends noted for total solids.

Phosphorus

Phosphorus is the element most pointed to when eutrophication of a body of water is discussed. It is thought that in most cases of eutrophication, if the amount of phosphorus entering a body of water can be controlled, the rate of eutrophication can be controlled. Because phosphorus is a common element found in domestic sewage, different types of industrial waste, combined sewer overflow and stormwater runoff, it was felt that measurement of this element was warranted.

In Appendix V-A, Tables A25, A26, A27, and A28 represent the data generated during the study of this parameter. The units used in measurement are mg/l of phosphorus. During the 1971 dry and wet weather periods, the ortho form of phosphorus was measured. Determinations of phosphorus during the 1973 and 1974 wet weather monitoring period were made on the total amount of phosphorus in the sample.

The mean concentrations of orthophosphorus during 1971 at the Points B and C for the dry weather monitoring period were very similar. The arithmetic means of the data for these two areas during the 1971 dry weather monitoring period were 0.16 and 0.20 mg/l, respectively. The mean value at Point A for this period was 0.02 mg/l. This shows a definite effect of encroachment of Lake Michigan waters into the studied reach. As reflected by the low values, the effect of encroachment is to dilute the phosphate concentration in the river water at Point A. Seasonal trends in this parameter may exist in the test reach but due to the effects of the lake, which vary with wind direction, these trends may be concealed.

During storm events of 1971, there was an increase in this parameter at Point A but a general decrease at the other two points (see Table A26).

There are probably two reasons why this parameter decreased at these two points during the storm events. First, the total volume of water passing a given point increased and had a dilution effect. Second, and probably the more important reason for a decrease in this parameter at the two points was the increase in seston content of the water (see Solids and TOC), which would tend to sorb this form of phosphorus out of solution. Data for Point B during day-1 of a storm, especially during the first six hours of a storm. This is most likely due to the first flush phenomena of combined sewers occurring during this time and increases the seston load in the river. Although data for Point C is scant, the same pattern seems to develop as was demonstrated at Point B.

Due to the kinetics of the seston - orthophosphate interaction, the decision was made to measure total phosphorus during the 1973 and 1974 wet weather monitoring program. Because of this decision there is no way dry and wet periods of 1971 and 1973 and 1974 weather periods can be compared.

The 1973 wet weather data for total phosphorus is presented in Table A27. It shows that during the first six hours of the collective events there was a considerable amount of total phosphorus at Point A, about 200% more than was at either of the other two monitoring points. When a comparison of means is made (Table 58) for day-1 (0-24 hours) of the 1973 wet weather events, no difference between the three monitoring points is noted. For day-2 (25-48 hours) comparison of this parameter, both Point A and Point C are similar to each other, as well as to day-1 of the monitored events; Point B shows a decrease in this parameter from day-1 to day-2 of the monitored events. During day-3 (49-72 hours) of the 1973 storm monitoring period, there is an increase at Point C when compared to days-1 and-2, although this increase is not statistically significant. On day-3 of these events Point B also shows an increase, although the magnitude of this increase is not as great as the increase indicated at Point C.

TABLE 58. COMPARISON OF TOTAL PHOSPHORUS YEARLY MEANS AT THE THREE RIVER SAMPLING POINTS

mg/1

Hours After Overflow Began	Point A		Point B		Point C	
	1973	1974	1973	1974	1973	1974
0-6	0.63	0.33	0.31	0.30	0.34	0.28
0-24	0.31	0.23	0.31	0.33	0.33	0.33
25-48	0.33	0.24	0.26	0.32	0.31	0.37
49-72	0.26	0.23	0.29	0.28	0.43	0.37

It is felt that the increase on day-3 at these two points is due to runoff entering the test reach from farther upstream. Day-3 of the 1973 events at Point A shows a decrease from days-1 and-2 at this site; this is probably due to dilution caused by the encroachment of Lake Michigan in the area.

Data gathered during 1974 (see Table A28), show a decrease in this parameter during the first six hours of the storm events at Point A when compared to the 1973 data. Data gathered at the other monitoring points during this period (0-6 hours after the start of a storm) is similar to the 1973 data at the respective sites. The decrease in this parameter at Point A is thought to be caused by (1) the dilution effect of the lake and/or (2) removal of phosphorus by the treatment sites.

Comparison of data at Point A for days-1, -2 and -3 during 1974 to the other two points and to the 1973 data shows a decrease in this parameter while it remained esentially the same at the other two monitoring points. Again, as in 1973, statistical comparisons of the data did not reveal any significant differences.

pH

Tables A29-A32 in Appendix V-A contain the pH values obtained at the three monitoring points on the Root River. The values obtained indicate that for the entire monitoring period a highly buffered and therefore very stable system exists in this reach of the Root River. The highest recorded pH value was 8.60 and the lowest value was 6.65. Based on the yearly means the range between the maximum and minimum values was not more than 0.2 pH units. This relationship generally held true for the individual storm events.

Data for this parameter during 1971 showed a general increase from day-1 to day-3 of a given storm event; just the opposite of what occurred during the 1973 monitoring season. During 1973, there was a decrease in this parameter at a given point over the three consecutive days of monitoring. During 1974, the trend was similar to that of 1971; that is, there was a general decrease in this parameter from day-1 of storm events through day-3.

In terms of water quality, pH variations in a range of 6.7 to 8.6 with extremes of 6.3 to 9.0 will support fish and other forms of aquatic life without problems. The permissible range of pH that a given organism can tolerate depends upon many other factors such as dissolved oxygen, temperature and prior acclimatization. The Aquatic Life Advisory Committee of ORSANCO (1955) considered changes in pH between a range of pH 5 to pH 9 to be unharmful to fish (25). In view of these recommendations, pH fluctuation in the Root River may be considered as a noncritical factor in terms of the river's biota.

Total Organic Carbon

The measurement of total organic carbon is one method used to determine the level of organic enrichment of a water body. The direct measurement of carbon can be used to replace the more time-consuming BOD and COD determinations.

Tables A33 through A36 in Appendix V-A contain data generated for this parameter during the four major monitoring periods. The baseline data gathered during 1971 dry weather monitoring indicated a higher organic content upstream at Point C with decreasing values downstream. This reduction in carbon content in the downstream portion of the test reach was maintained during the 1971, 1973 and 1974 wet weather events. The depression of the parameter as one proceeds downstream is probably caused by mixing of lake and river waters.

Comparisons of data over sampling periods and over points were done using one of two statistical procedures. The statistical techniques applied were either an analysis of variance test or the Student t Test.

Analysis of dry weather data obtained during 1971 indicates that there was a significant difference between all three points. The mean of all dry weather measurements taken in 1971 for each point is as follows:

Point A, 11 mg/l
Point B, 17 mg/l
Point C, 22 mg/l

Means were determined for the first twenty-four hour sampling period for all storms monitored during 1971 (Table 59); comparison of the 1971 wet and

TABLE 59. TOC YEARLY MEANS
(mg/l)

	Hours after overflow began						
Year	0-6	7-12	13-18	19-24	0-24	25-48	49-72
1971 Dry					11		
1971 Wet	10	9	8	8	9	10	9
1973	12	12	12	12	12	11	13
1974	14	14	15	15	15	14	12
1971 Dry					17		
1971 Wet	15	15	15	14	14	14	16
1973	17	17	16	16	16	14	14
1974	18	18	18	20	19	18	17
1971 Dry					22		
1971 Wet	15	28	27	18	20	19	19
1973	15	14	41	15	15	15	18
1974	17	18	16	16	18	19	18

dry weather events shows that there is about a 2 mg/l reduction in the amount of total organic carbon for 1971 dry weather to 1971 wet weather events.

This reduction was analyzed at all sites using the Student t Test and was found not to be significant. The 1971 wet weather data were analyzed over all points and indicated that a significant difference existed between them. When point was compared against point, there was a significant difference indicated between Point A and Point B, but not between Point B and Point C.

Analysis of 1973 data indicated that there was a significant difference between Point A and Point B but no significant difference between Point B and Point C. Storm data for this parameter during 1973 had higher values than the 1971 storm data. Comparison of each site for each of the two different storm periods using the Student t Test showed that there were no significant differences between these two periods at Point C or Point B but that there was at Point A.

When 1974 data was compared for all three points, no significant differences were observed for day-1 or day-2 data. Differences which were significant were found for day-3 data. Comparison of point to point for 1974 showed no significant differences for day-1, -2, or -3. The data show an increase in this parameter at all three points from 1973 to 1974 although the increase observed is not significant at any of the monitoring points.

Biochemical Oxygen Demand

The biological community which inhabits a stream is dependent on the amount of dissolved oxygen available in the water to carry on respiratory functions and these populations place a demand on the oxygen available. Through the normal aeration processes (photosynthesis, turbulence, and diffusion) of streams this need is met and the amount of oxygen present is generally greater than the demand. When organic material is contributed to a water body, especially in sewage, it acts as a food source for the decomposer portion of the ecosystem. This extra food promotes growth of this population (the decomposers) which in turn will demand more oxygen for respiration. This oxygen demand is normally expressed in terms of biochemical oxygen demand (BOD).

As such, BOD is not a measurement of a specific substance but is rather a measure of the total impact of the various organic compounds which enter a body of water and exert an oxygen demand. This impact is based on the amount of oxygen needed for the breakdown of these organic compounds by the population of decomposer organisms and those compounds which utilize oxygen directly. It can only be used as an indicator of water quality and to detect areas of pollution. The nature of the BOD test lends itself to detecting areas in a stream where a potential decrease in the amount of dissolved oxygen may occur.

Tables A37, A38, and A40 in Appendix V-A contain the data gathered during the four major monitoring periods for this parameter.

Data for the 1971 dry weather period show that the water quality at Point A, as determined by this parameter, is better than at the other two monitoring points. The mean for this period at Point A was 2 mg/l with a maximum value of 10 mg/l and a minimum value of 1 mg/l. The maximum value, 10 mg/l, only occurred once and due to the type of environment being examined at this point, there is some doubt to the validity of the measurement; the next highest value at this point was 3 mg/l. Measurement of this parameter at Point B had a mean value of 5 mg/l and maximum and minimum values of 8 mg/l and 2 mg/l respectively. Point C had the highest BOD mean value of the three monitoring sites, 7 mg/l. The maximum value obtained at this point was 20 mg/l and the minimum value was 1 mg/l.

BOD values, as judged by the mean at each point for the 1971 storm events (see Table B29), indicate that during a majority of the storm events there was no change in this parameter above background at Point A and Point B. The measurement of this parameter at Point C shows an increase above the background of 1 mg/l on day-1 of the storm events, as is evident from a comparison of the means. The mean, maximum and minimum at each point for this period are listed below for day-1 of the storm events:

	BOD values (mg/l)		
	Mean	Maximum	Minimum
Point A	2	4	<1
Point B	5	9	2
Point C	8	20	1

The BOD mean values for the 1973 storm season indicate a marked improvement in water quality at Point C (Table 60). The mean value for day-1 of the storm events was 3 mg/l. The range of values at this point was from 1 mg/l to 6 mg/l. Values at Point B also improved in 1973 when compared to the 1971 values. The mean, minimum and maximum values at this point are as follows: 5 mg/l, <1 mg/l, and 13 mg/l. Values at Point A show little, if any, change. The mean value for day-1 at this point was 3 mg/l with a minimum of <1 mg/l and a maximum value of 8 mg/l.

BOD measurements during 1974 wet weather events showed a slight increase when compared with the 1973 measurements. The variations in this parameter at Points A and B during 1974 were similar (see Table 60). The variation at Point C was similar to that of the other two points with the exception of Run No. 43 (see Table A40). The mean values for the three days of monitoring along with the minimum and maximum values for each point during 1974 are listed below.

Table 60 presents the mean values for all wet weather events monitored during 1971, 1973, and 1974 for the four 6-hr monitoring periods and the mean values for day-1, -2, and -3 of the same. From the data, it can be seen that the Root River is a moderately polluted river (26).

TABLE 60. BOD YEARLY MEANS, mg/l

Hours	0-6	7-12	13-18	19-24	0-24	25-48	49-72
Point A							
1971	3	2	2	2	2	3	3
1973	2	3	3	3	3	3	3
1974	4	4	3	4	3	3	3
Point B							
1971	6	5	5	4	5	5	4
1973	4	5	4	5	5	3	3
1974	5	4	4	4	5	5	4
Point C							
1971	5	7	4	8	8	6	6
1973	3	3	3	3	3	3	3
1974	5	5	5	4	5	5	5

Fecal Coliform Bacteria

In this phase of the investigation fecal coliform bacteria were utilized as an indicator organism. The indicator organism concept states that, under a given set of physical and/or chemical environmental conditions, certain life forms will exist and therefore are an indication of the conditions that exist in the water body at that time. The presence of fecal coliform bacteria in the fresh water environment is an indication that an addition of fecal material to the waterbody has recently occurred.

The organisms are introduced into a water body from a variety of different sources. These include such sources as discharges from domestic sewage treatment plants, storm runoff from natural areas, and runoff from feed lots.

The Wisconsin Department of Natural Resources (27) states that water related recreation which is classified as secondary contact recreation should not be carried out on waters which have a fecal coliform count exceeding 2000 organisms per 100 ml as an average. That report classifies secondary contact recreation as that type of recreation which does not involve significant risk of ingestion. It states further that on waters "specifically" designed for recreation, the geometric mean of fecal coliform bacteria should not exceed 1,000 organisms per 100 ml as an average. For recreation areas where primary contact is likely, such as swimming, the geometric mean of the samples should average no higher than 200/100 ml with less than 10% of the samples having a count exceeding 400/100 ml.

Tables A41 - A44 in Appendix A list the fecal coliform data gathered during the 1971 dry weather monitoring period, the 1971 wet weather monitoring period, the 1973 wet weather monitoring period, and the 1974 wet weather monitoring period, respectively.

When the data from the 1971 dry weather period is examined (Table A41) one finds that only the Point C fecal count is below the suggested 200 organisms per 100 ml and that the other two points have populations which were considerably above this. This observation leads to the conclusion that during dry weather periods the contribution of fecal coliform bacteria (or nutrients for their growth) from various sources was significant in the City of Racine. Comparing Point A to Point C indicated that a 400% increase in the number of these organisms occurred as the river passed through Racine during the dry weather monitoring period.

Storm data from 1971 indicates that during these events the counts of fecal coliform bacteria at all areas increased over the dry weather data. The general pattern observed was a rise in the fecal coliform count on day-1 of a storm which steadily decreased over day-2 and -3 of a monitored event. The geometric mean of this group at Point C over the entire monitoring period was approximately 200 organisms per 100 ml. Point A had the highest total number of these organisms on day-1 of a storm event of all the points, with a geometric mean of 5986 organisms per 100 ml. At this point the decline, as measured by the geometric mean for the year, from day-1 to day-2 and from day-1 to day-3 was 64% and 77%, respectively. Point B had a geometric mean for day-1 of all storm events of 1775 organisms per 100 ml, 30% lower than the value for Point A. This difference would indicate that the pollution load in the river is less at Point B than at Point A. The decrease in this parameter over the second and third day of the sampling when compared with day-1 of an event is 30% and 57%, respectively, at Point B. One reason that the pollution load falls off so rapidly at Point A may be due to the effect of encroachment in the area of Lake Michigan. This effect would be to dilute the waters in the area and create the illusion that a decrease in this measured parameter happened faster here than at Point B.

Table A43 lists the data generated for this parameter during the 1973 storm season. Data gathered at Point A during this monitoring period indicated conditions which existed after the treatment sites went into operation. The geometric mean at Point A for day-1 of all discharge events was 3084 organisms per 100 ml; 497 less than the number of organisms recorded for the storm period, day-1 of 1971. This reduction in total number may be due in part to the operation of the two treatment sites in the area.

The reduction in numbers of this organism noted at Point A was not followed by a similar decrease in this parameter at the other sites.

Fecal coliform data at Point B showed almost a 20% increase from 1971 and 1973 and a 225% increase at Point C during this time. Another factor which may have a strong influence on the reduction of fecal coliform bacteria at Point A was the higher lake level in 1973 than in 1971. The higher lake level could have influenced this parameter by causing more dilution.

Fecal coliform data gathered during 1974 showed a reduction in the bacterial populations, when compared with 1973 data at Point A. Data gathered at the other two monitoring points in 1974 were similar to the data obtained during the 1973 sampling effort.

The difference noted at Point A from 1973 to 1974 was not statistically significant due to the amount of variation in the data both years, but the decrease in this parameter at this point was about 60% on day-1 of wet weather events, 50% on day-2 and 20% on day 3. This decrease was not noted on day-1 of wet weather events at Point B, although there was a decrease of 27% and 29% during day -2 and-3, respectively, at this point. The decrease at Point A may be due in part to the continued rise in the level of Lake Michigan, which occurred over the entire duration of the project, and in part, to the effect of the treatment systems on this parameter. In addition, there appears to be a substantial decrease in the fecal coliform concentration during treatment events in which chlorination was performed at Site II as compared to events during which chlorination was not performed.

River Sediment Chemical Analyses

River sediment for chemical analysis were collected once during 1971, four times during 1973, and twice during 1974. Samples for these analyses were collected from cross-sections of the river at all three of the major monitoring points. Samples at each site were collected at quarter points across the river; these samples were then composited into a single sample for each point.

Samples of bottom muds were taken using a Wildco-Ekman grab sampler with a 15.2 cm (6 in.) square sampler jaw opening. This sampler is designed for use in soft, finely divided littoral bottoms of lakes, ponds, and streams which have little vegetation and intermixtures of sand, gravel and other coarse debris. The sampler is primarily used for quantitative and qualitative sampling of microscopic bottom fauna as well as obtaining, as in this case, samples of bottom materials for chemical analysis.

The Ekman grab sampler is operated by opening the jaws and settling the activation springs over their respective retaining bars. The sampler is then lowered to the bottom of the area to be sampled. To prevent loss of materials which may have been disturbed by the impact of the sampler with the bottom, it is raised, via the attached rope, about 0.3 m (1 ft) off the bottom and moved upstream of the initial impact point where it is allowed to settle freely into the bottom material. A weighted messenger is then send down the attached line, which upon impact, activates the springs on the sampler causing it to close, thus taking a sample. To prevent loss of material upon retrieval from washout, the device is equipped with lids which are kept closed by water pressure. Sample size is variable, dependent on bottom composition, but generally is in the range of one to two liters per grab.

Samples used for chemical analysis were collected directly from the dredge and put into sample bottles marked for chemical analysis.

After all samples for a given date had been gathered, they were taken back to the laboratory for analysis of the following parameters:

Kjeldahl nitrogen
Ammonia nitrogen
Total phosphorus
Total solids
Volatile solids

Analyses were performed according to the methods given in Appendix IV-D. Table 61 lists the results of these analysis for the respective sampling dates.

Statistical analysis of the data was performed for each of the parameters using an analysis of variance test to compare points over time. Results of these tests reveal there was no significant change over time at any of the points for ammonia nitrogen and Kjeldahl nitrogen. Analysis of total solids, volatile solids and total phosphorus showed no significant difference over time, but there was a significant difference indicated between Point C and the other two sampling sites. The data generated from these samples indicate that, because of the high nutrient content, the potential exists for nutrient loading of the river from the sediments. With the exception of total solids which increased from Point A toward Point C, all parameters measured increased from Point C to Point A.

In addition to running chemical analyses on the bottom muds from the river, analysis of different points in the water column was undertaken to determine what, if any, nutrient profile existed. These analyses were taken using the Bacon water sampler. The Bacon water sampler is a mechanically operated piston cylinder device which selectively takes approximately 500 ml of water at any chosen level. The sampler is lowered to a selected level and a trip rope, controlling piston activation, is pulled moving the piston plug upward and opening the sample chamber. The liquid rushes in discharging air through the top of the cylinder by route of the piston shaft discharge point. Once the discharged air bubbles reach the surface, the person sampling releases the piston control rope allowing the piston plug to settle back into place closing the sample chamber. The sample is then drawn to the surface and its content discharged into a marked collection bottle for identification and analysis. No preservatives or chemicals were added to the water samples collected in this manner. One composite sample was drawn at each dredge sample location.

Samples were collected at six different sites in the test reach of the river. These were discrete samples taken from top to bottom in 0.9 m (0.3ft) intervals. Table 62 contains the data generated from these samples. An increase is noted in each parameter proceeding upstream from Point A toward Point C.

TABLE 61. ROOT RIVER BOTTOM SEDIMENT ANALYSES *

	15 June 1971	20 April 1971	27 June 1973	22 August 1973	15 October 1973	30 April 1974	11 July 1974
Point A							
Kjeldahl nitrogen	1600	3268	10,023	1947	3618	1525	818
Ammonia nitrogen	124	55	185	59	197	66	57
Total phosphorus	623	1048	1224	597	828	408	201
Total solids	39.9	50.3	41.8	54.6	42.8	39.2	37.2
Volatile solids	4.0	6.4	3.4	3.5	3.7	3.8	3.8
Point B							
Kjeldahl nitrogen	1600	9583	1944	3228	4418	1014	765
Ammonia nitrogen	73	72	28	128	218	47	21
Total phosphorus	630	757	1029	936	995	289	143
Total solids	31.5	40.8	52.4	38.8	39.3	59.3	38.7
Volatile solids	4.5	4.5	3.8	4.0	3.8	2.7	2.7
Point C							
Kjeldahl nitrogen	--	50	500	211	2982	1608	622
Ammonia nitrogen	--	0	3	15	185	123	41
Total phosphorus	--	94	36	25	1080	478	91
Total solids	--	77.5	79.7	80.6	44.4	49.4	68.2
Volatile solids	--	0.7	1.0	1.2	3.1	4.0	2.5

* **Note:** Nitrogen and ohosphorus values are expressed in milligrams per kilogram of wet mud, total solids and volatile solids as percent by weight of a wet mud sample.

TABLE 62. CONCENTRATION OF ORTHO PHOSPHATE AS P AND NITRATE AS N MEASURED AT SIX RIVER SITES (mg/1)

Depth Meters	Depth Feet	Main St. Bridge Point A	Point B State St. Bridge	Marquette St. Bridge Point C	Above Point B Memorial Drive Bridge	Below Horlick Dam Point C	Above Horlick Dam Point C
				Ortho Phosphate as P			
0.0	0	0.38	0.17	0.15	0.32	0.53	0.53
0.9	3	0.18	0.17	0.20	0.35		0.53
2.7	9	0.17	0.12	0.20	0.35		
3.7	12	0.19	0.55				
4.6	15	0.55	0.35				
5.5	18	0.35					
				Nitrate as N			
0.0	0	0.16	0.40	0.65	0.65	0.70	0.65
0.9	3	0.16	0.25	0.73	0.55		0.90
1.8	6	0.06	0.25	0.60	0.78		
2.7	9	0	0.45				
3.7	12	0.08	0.35				
4.6	15	0.15	0.35				
5.5	18	0.58					

Benthic Macroinvertebrates

The benthic population found in a stream is made up of a diverse collection of fauna and flora. It includes such things as protozoans, algae, rotifers and macroinvertebrates. By definition, the benthic population is that community of organisms which live on or in the bed of aquatic or marine systems. They can also be found attached to aquatic plants and other substrata found on or near the stream bed. The portion of this community examined during this study were macroinvertebrates.

The species groups, as well as their numbers, which make up this benthic community will vary with the physical and chemical conditions of the stream. Patrick (20) points out that in oligotrophic streams where the nutrient level is very low, there exists a population which has extremely high diversity but extremely low population levels. The physical and chemical factors of such a stream are those of very low nutrient levels. The reverse is true for streams which have a high nutrient level.

Sources of energy and nutrients for the benthic communities of streams are quite different than for either the lake or terrestial communities found in the same types of niche. The total nutrient load of a stream is continuously being replaced from the upstream direction and may be of three separate types: dissolved, suspended, and organismal. In many lake systems it has been determined that nutrients are generally in a recycling type of system, while in a stream, because of its flowing nature, nutrients are being renewed continually. If this were not the case, the benthic community of a would be divided into different levels of organism or energy transfer levels, the decomposers, primary producers, herbivores, and one or two levels of carnivores.

The structure of the macroinvertebrate community is commonly used as an indication of conditions in both polluted and unpolluted streams. The prime reason that this population has been used almost exclusively as an indicator rests in the fact that the life cycles of many of the benthic species are long and that this group is primarily confined to the bottom area. If a perturbation to that system occurs which lasts for only a short time and it is toxic to the indicator organisms, the investigator who works with the system can discern that something has happened even if he cannot detect it through physical or chemical means. Because of this, much research has been carried out to try and classify benthic organisms according to their pollution tolerance. A system of this type allows the use of these organisms as criteria of pollution (29). Presently, a number of investigators have used indices developed from information theory that expresses species diversity to summarize the community structure of benthic organisms. The use of the diversity index is considered as one of the best and most sensitive indicators of ecological change (30), (21). The first use of the species diversity methods occurred in 1966 when investigations to examine the effects of organic enrichment on streams was carried out by Wilhm and Dorris (32).

Samples for biological classification and identification were collected from the river cross-sections at each of the three major sampling points.

Characterization of each point was done by taking quarter point samples along transects at each point. Samples from the bottom were taken using a Wildco-Ekman dredge (previously described).

The method of bottom fauna collection involves collection of a bottom sample which is then drawn to the surface. Before the dredge is lifted above the water line, it is placed into a five gallon plastic wash bucket which has had the bottom replaced by a 30 mesh screen reinforced by 1.27 cm (1/2 in.) hardware cloth. The dredge and bucket are then raised slightly above the water line and the contents of the dredge are discharged into the bucket. The sample in the collection bucket is then swirled lightly in the water washing away silt and mud and leaving the benthic organisms and larger detritus particles. This material is then washed into a collection bottle containing a 70% alcohol solution and taken back to the laboratory for identification.

Data comparisons were made using a diversity index, developed by Cairns (33), over sites for a given sampling date and over dates for a given sampling point. Data for these studies is presented in Table 63.

The diversity indices generated using data obtained from samples taken at Point A indicate an extremely degraded and unstable system. The biota found at this point were represented by very few toxa; during the June 1974 sampling effort at this point, only one type of organism was noted, this being the species Tubifex, an indicator of an organically enriched system.

As measured using the Cairns index, Points B and C both were in a healthier state than Point A. Point C remained fairly stable through the monitoring program with only one exception, that being the April 1974 sample which was probably influenced by the scouring actions of the Spring high flow period. Point B remained at a somewhat degraded level throughout the monitoring period.

Light - Dark Bottle Test

The light-and-dark bottle technique may be used as a simple means of measuring the total diurnal metabolism of a body of water. It is also a starting point to be used for charting energy flow through an aquatic system.

Light-and-dark bottle tests were conducted at all three river sites in triplicate on August 13, 1973 and August 30, 1974. The purpose of these tests was to indicate the potential daytime peaks and nighttime troughs in the river's dissolved oxygen concentration attributable directly to photosynthetic activity. These tests, by design, exclude anchored vegetation, and concentrate on organisms and/or particles contained in the water column.

Baseline dissolved oxygen determined immediately upon collection at 9:00 am on August 13, 1973 was found to be 5.0, 5.4, and 7.3 mg/l at the Point C,

TABLE 63. BENTHIC ORGANISM SURVEY

Date of Survey		Point A	Point B	Point C
April, 1973	Number of organisms	253	135	9
	Number of species groups	3	2	5
	Predominant species group	Tubifex	Tubifex	None
	Cairns Index Number	0.020	0.05	0.89
June, 1973	Number of organisms	250	55	44
	Number of species groups	1	5	8
	Predominant species group	Tubifex	Tubifex	Ephemeroptera
	Cairns Index Number	0.000	0.20	0.63
August, 1973	Number of organisms	150	150	36
	Number of species groups	1	6	7
	Predominant species group	Tubifex	Tubifex	Ephemeroptera
	Cairns Index Number	0.000	0.10	0.81
October, 1973	Number of organisms	300	300	300
	Number of species groups	7	16	14
	Predominant species group	Tubifex	Tubifex	Tubifex
	Cairns Index Number	0.07	0.40	0.44
April, 1974	Number of organisms	250	275	300
	Number of species groups	4	8	6
	Predominant species group	Tubifex	Tubifex	Tubifex
	Cairns Index Number	0.006	0.26	0.004
June, 1974	Number of organisms	275	275	269
	Number of species groups	1	13	8
	Predominant species group	Tubifex	Tubifex	Ephemeroptera
	Cairns Index Number	0.000	0.33	0.66

B, A, respectively. The Winkler method of oxygen determination was used. The remaining samples were placed in the direct sunlight at a central location and left until 2:00 pm. All were then analyzed for dissolved oxygen concentration. The data is contained in Table 64.

TABLE 64. LIGHT-AND-DARK BOTTLE DISSOLVED OXYGEN DETERMINATIONS (1973) (mg/l)

Point C	Initial DO Concentration	DO Light Bottles	DO Dark Bottles
Test 1	5.0	9.7	4.3
Test 2	5.1	9.9	4.2
Test 3	5.0	9.6	4.3
Mean	5.0	9.7	4.3

Point B	Initial DO Concentration	DO Light Bottles	DO Dark Bottles
Test 1	5.3	8.5	4.7
Test 2	5.4	8.5	4.8
Test 3	5.6	8.5	4.7
Mean	5.4	8.5	4.7

Point A	Initial DO Concentration	DO Light Bottles	DO Dark Bottles
Test 1	7.3	7.1	7.1
Test 2	7.4	7.0	7.0
Test 3	7.5	7.1	7.0
Mean	7.4	7.1	7.0

Algal activity was indicated at Point C and to a slightly lesser extent at Point B. The relative increases and decreases were similar with a slight overall decrease in magnitude apparent at Point B, probably due to a reduction in concentration of photosynthetic material. There was apparently no algal activity at Point A. This is consistent with 1971 data where extremely limited numbers of algae were reported. A slight oxygen demand is indicated regardless of time of day.

Data obtained on August 30, 1974 indicated a baseline dissolved oxygen concentration of 4.5, 2.1, and 6.1 mg/l for Points A, B, and C, respectively. Baseline data was obtained at 5 am on the above date. As in 1973, the Winkler method of oxygen determination was used. At the time that baseline conditions were determined, triplicate light and dark bottles were set up at each point and incubated at a depth of 15 cm (6 in.) for six hours. After the incubation period, oxygen determinations were run on all samples, data from these determinations is presented in Table 65.

Table 65. LIGHT-AND-DARK-BOTTLE DISSOLVED OXYGEN DETERMINATIONS (1974) (mg/l)

Point C

	Initial DO Concentration	DO Light Bottles	DO Dark Bottles
Test 1	6.1	5.3	5.9
Test 2	6.0	5.5	5.4
Test 3	6.1	5.8	6.2
Mean	6.1	5.5	5.8

Point B

	Initial DO Concentration	DO Light Bottles	DO Dark Bottles
Test 1	2.3	2.7	1.9
Test 2	2.0	2.5	1.7
Test 3	2.0	2.8	2.1
Mean	2.1	2.7	1.9

Point A

	Initial DO Concentration	DO Light Bottles	DO Dark Bottles
Test 1	4.6	4.8	4.2
Test 2	4.3	4.7	4.5
Test 3	4.6	4.9	4.3
Mean	4.5	4.8	4.3

Algal activity was indicated at Points A and B while there was a net loss of oxygen at Point C which would indicate a slight oxygen demand at that point.

Biological Characterization

During the early summer of 1971, a special study was undertaken at Point C to characterize it biologically. Table 66 is a summary of the results.

TABLE 66. SPECIES CLASSIFICATION AT HORLICK SITE (1971)
[150 m (500 ft) DOWNSTREAM OF DAM]

Algae

- *Spirogyra*
- *Mougeotia*
- *Zygema*
- *Chladophora*
- *Microspora*
- *Scenedemus*
- *Ankistrodesmus*
- *Volvox*

Protozoans

- *Ichthyophthiririus*
- *Spirostomum*
- *Nassula*
- *Anisonema*
- *Frontonia*
- *Englena*
- *Paramecium*

Crustaceans

- *Cambarus* sp. (common Crayfish)
- *Diaptomus*

Mollusco

- *Strophitis* sp.
- *Sphaerium* sp.
- *Pleurocera* sp.

Miscellaneous Invertebrates

- *Tubifex*

Fishes

Lepomis marchirus	(common bluegill)
Notropis sp.	(common shiner)
Ictalurus sp.	(bullhead)
Exos luelus	(Great Norther Pike) Rare
Salmo gairdneri	(Rainbow Trout) Spring only
Cyprinus carpio	(Carp)
Castostomus conmerson	(connom White Sucker)
Moxostoma macrolepiodotum	(common Norther redhorse)

The samples collected below the Point C indicate a bottom configuration characterized by rock and shale rubble interspersed with small patches of detrital materials. It was found to be ideally suited for harboring diverse speciation and proved to be biologically the richest area sampled within the test reach. Vast mats of Cladophora and Spirogyra interspersed with Mougeotia provided a good deal of natural cover and food for the herbivorous fauna population. All of the organisms identified in Table 66 were found at Point C. Several variables were observed, however, which limited the diversity of this point's ecosystem and determined the species dominance at the point. The water samples taken at Point C were nutritionally rich, especially in nitrogen, phosphorus, and carbon, which probably contributed significantly to the high chlorophyll concentrations observed throughout the period. The combination of nutritionally rich water and the shallow depth of the river at this point, generally less than 0.3 m (1 ft), resulted in ideal conditions for the culturing and growth of algae and bacteria.

Special Analysis

Several special tests were conducted during the monitoring period to explore in more detail areas of concern uncovered by the established battery of tests. Among these were BOD determinations, sedimentation tests, sieve analysis in relation to sedimentation, and pesticide analysis.

Biochemical oxygen demand appears to remain unchanged at all three sites despite the input of storm-generated discharge into the test reach. This observation was a cause for concern from its discovery of the phenomenon early in the monitoring period. A BOD series of 3, 5, 10, 15 and 20 days was run on the sample taken for Run No. 6 (8/10/74) at Site A (Main Street Bridge). This test revealed a very normal BOD curve and indicated there was no delay in BOD expression in the river. The possibility was explored that the discharge plume was bypassing the sampling point. A series of BOD determinations were made from the river cross-section at point A and again proved inconclusive (Table 67). There is a possibility that the continuous demand in the river during normal flow conditions is sufficient to mask any additional BOD input as a result of storm-generated discharge. Determinations to uncover the validity of this possibility have not yet been made.

Suspended solids, being one of the parameters which exhibited some change as a result of a rainfall occurrence, was explored further to ascertain the possibility of sedimentation within the river. Both theoretical calculations and field measurements were done.

Using the formula, $V = (1,486)\ (R^{1/6})\ (K[S_s - K]\ d/N)$ taken from the Storm Water Management Model (35), a critical velocity to retain 50% of the estimated combined sewer input suspended solids (see particle size distribution following) in suspension was calculated. Using values of 0.035 for Manning's coefficient; an R taken from the river cross-section at Site B; and the (K) and (S_s) values from the math model; and inserting the (d) value equal to or less than 50% of the analyzed particle size, a calculated critical velocity of 0.168 m/sec (0.553 ft/sec) was obtained. The eight year average river flow at the Horlick Dam is 221 cu m/min (130 cfs). Based on this flow and a cross-sectional area of 151.0 sq m (1,625 sq ft)

TABLE 67. BOD VALUES FROM RIVER CROSS-SECTION (1971) (mg/l)

During the period of overflow occuring as a result of Storm 12, 11/18/71, cross section surface BOD's were drawn at the Main Street Bridge. Starting on the North Bank and proceeding south, samples were drawn from the surface at 3.05 M (10 ft) intervals for a total of 12 samples. All samples were analyzed for TOC and TIC while only the odd numbered samples were analyzed for BOD. The resultant values are shown in the table below.

Distance from North Bank					
Meters	Feet	TC	TIC	TOC	BOD
0	0	37	24	13	4
3.05	10	33	22	11	
6.10	20	32	22	10	2
9.15	30	31	22	9	
12.20	40	38	24	14	3
15.25	50	31	22	9	
18.30	60	31	22	9	3
21.35	70	32	22	10	
24.40	80	33	23	10	4
27.45	90	33	22	11	
30.50	100	32	22	10	7
33.55	110	33	21	12	

at Point B, a river velocity of 0.024 m/sec (0.08 ft/sec) can be calculated for Point B. When compared to the critical velocity, sedimentation is indicated.

To verify downstream sedimentation, sedimentation-collection buckets were placed at both the Main Street and Western Publishing Sites. The results of the collections and later analysis for 1971 Storm Nos. 8, 9, and 11 are contained in Table 68. There is indication that sedimentation occurs and that it is affected by both wind direction and velocity. An upstream wind (Storms 9 and 11) appears to cause a higher sedimentation rate and the majority of the increase appears to be in nonvolatile solids.

During 1974, sedimentation characteristics of the river were determined after a prolonged dry period had occurred, but prior to any discharge. Sedimentation collection equipment was installed at each of the three monitoring sites and left suspended 0.5 m (18 inches) off the river bottom for a total of 14 days. This is in accordance with the method described by Edmondson (36). The rate of sedimentation at each point during dry weather flow was estimated to be 7.0×10^{-4} g/m^2/day at Point A, 7.3×10^{-4} g/m^2/day at Point B and 3.8×10^{-4} g/m^2/day at Point C. These data appears to confirm the facts established by the chlorophyll analysis and the sedimentation test run

TABLE 68. SOLIDS COLLECTED IN RIVER SEDIMENTATION COLLECTION BUCKETS (g/day/sq m)

Storm	Date	Total Solids Point A	Total Solids Point B	Volatile Solids Point A	Volatile Solids Point B	Wind Direction Avg.	Average Wind Velocity, km/hr
008	9/20/71	7.4	2.9	62.0	2.9	35°	40.2
009	9/27/71	18.7	2.8	7.6	2.8	89°	19.3
011	11/1/71	10.28	13.4	15.5	6.1	60°	48.3

during 1971, that there is a loss of energy near Point B and the subsequent settling out of solids at this point.

Chlorophyll Analysis

During August 1971, two chlorophyll profiles were determined on the lower Root River at five selected test sites. One uniform composite of the entire water column was collected. The resultant chlorophyll concentrations were determined using an analytical method developed by Yentsch and Menzel in 1963 (37). This method consists of filtering a measured volume of water into a glass fiber filter which is then extracted with an 85% acetone solution to free the chloroplast pigments. The extract after centrifugation is then placed in a fluorometer and fluorescence is measured using an excitation wavelength of 430 to 450 millimicrons. The readings of fluorescence values were related to a standard curve for conversion to parts per billion of chlorophyll and these values were used to calculate total kg of chlorophyll per one meter cross-section of each selected site. The results in total kg of chlorophyll per respective cross-section for two surveys in August 1971 are shown in Figure 59.

One observation made in 1971 from the August profile survey of chlorophyll, and supported by the additional readings available for the biological survey, was the tremendous decrease in total mass of chlorophyll between the location of Point B and the Memorial Street Bridge, which is located upstream from Point A

Pesticide Analysis

Samples for pesticide analysis were collected from the treatment sites and the River monitoring sites during 1971, 1973, and 1974. The months and sites sampled are listed below. All samples were taken at the beginning of a storm which was preceded by a long dry spell.

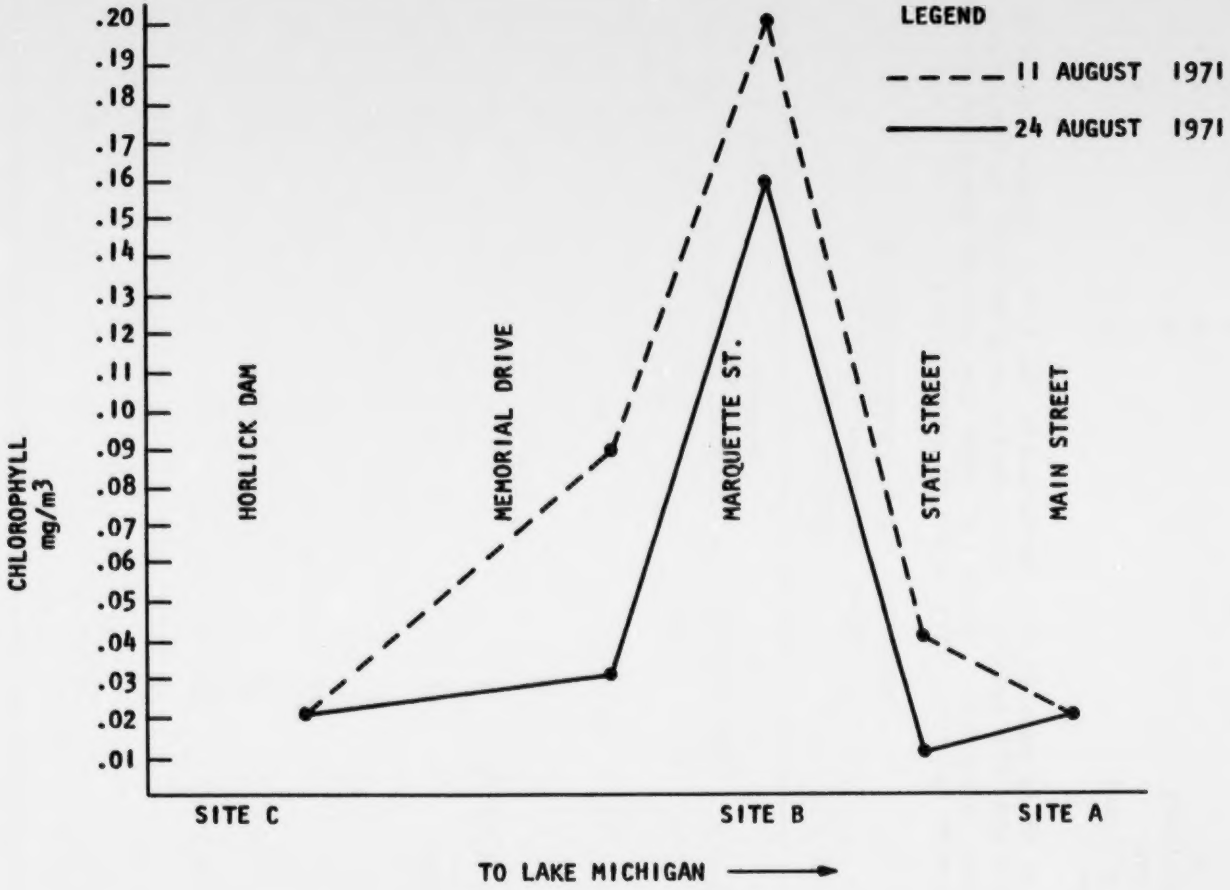


Figure 59. Lower Root River chlorophyll profile.

	September, 1971	September, 1973	July, 1974
Site A		x	x
Site B	x		x
Site C	x	x	x
Site II raw	x	x	x
Site II effluent		x	
Site IIA raw		x	x
Site IIA effluent		x	
Site I raw			x

Analysis of these samples were performed by the E.P.A. Region V Laboratory, on the 1971 and 1974 samples. 1973 samples were analyzed by Limnetics, Inc., of Milwaukee, Wisconsin. Results of these analyses are presented in Tables 69, 70, and 71 for 1971, 1973, and 1974 respectively.

TABLE 69. 1973 PESTICIDE RESULTS
Date: September 26, 1971
(ng/l or ppt)

Pesticide	Horlick Dam (River)	Western Publ. Co. (River)	Site II Raw
Lindane[a]	5	7	<1
Heptachlor	13	10	<1
Aldrin	16	7	14
Heptachlor Epoxide	12	15	16
Methozychlor	46	34	58
Dieldrin	<1	5	<1
Endrin	20	19	<1
o,p DDE	15	29	30
p,p'DDE - o,p-DDD	11	14	20
p,p'DDT	44	39	66
o,p-DDT	28	21	34

a. Analyses performed through Region V, EPA, using standard EPA procedure

TABLE 70. PESTICIDE RESULTS FOR STORM 13- AFTER LONG DRY SPELL
Date: September 4, 1973 (45 days since last overflow)
(in ng/l or ppt)

Pesticide[a]	Main Street (River)	Horlick Dam (River)	Site IIA Raw	Site IIA F. Effluent	Site II Raw	Site II F. Effluent
Endrin	<10	200	260	<10	100	100
Lindane	<15	50	90	230	130	230
Methoxychlor	<15	<15	<15	<15	<15	<15
Heptachlor	<10	<10	<10	<10	<10	<10
Aldrin	<10	<10	<10	<10	<10	<10
Dieldrin	<10	<10	120	<10	<10	<10
op' -DDT	<10	<10	<10	<10	<10	<10
pp' -DDT	<15	<15	<15	<15	<15	<15
pp' -DDD	<15	<15	<15	<15	<15	<15
Heptachlor epoxide	<10	<10	<10	160	<10	<10
op' -DDD+	<10	45	<10	<10	<10	60
op' - DDE+						
op' - DDE						

a. Analyses performed by Limnetics, Inc., Milwaukee, Wis. using EPA procedure of Federal Register, June 23, 1373.

TABLE 71. PESTICIDE RESULTS
Date: July 2, 1974
(ng/l or ppt)

	Point A	Point B	Point C	Site I raw	Site II raw	Site IIA raw
Total DDT	148.0	<1.0	50.0	61.0	89.0	5.0
o,p-DDT	73.0	<1.0	6.0	35.0	55.0	<1.0
p,p'-DDT	75.0	<1.0	15.0	<1.0	<1.0	<1.0
Total DDD	<1.0	<1.0	24.0	26.0	34.0	5.0
o,p-DDD	<1.0	<1.0	24.0	26.0	<1.0	<1.0
p,p'-DDD	<1.0	<1.0	<1.0	<1.0	34.0	5.0
Total DDE	<1.0	<1.0	5.0	<1.0	<1.0	<1.0
o,p-DDE	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0
p,p-DDE	<1.0	<1.0	5.0	<1.0	<1.0	<1.0
Methoxychlor	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0
Aroclor 1221	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0
Aroclor 1232	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0
Aroclor 1242	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0
Aroclor 1248	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0
Aroclor 1254	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0
Aroclor 1260	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0
Aroclor 1262	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0
Aroclor 1268	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0
Di-*n*-butyl phthalate	2500.0	1100.0	1500.0	2500.0	1400.0	8200.0
Diethylhexyl-phthalate	7800.0	6900.0	3500.0	2200.0	106,000.0	26000.0
Lindane	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0
Heptachlor	4.0	39.0	5.0	<1.0	<1.0	<1.0
Aldrin	6.0	<1.0	8.0	<1.0	<1.0	<1.0
Heptachlor Epoxide	5.0	4.0	2.0	32.0	23.0	<1.0
Dieldrin	10.0	<1.0	4.0	<1.0	14.0	<1.0
Endrin	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0

SECTION VI
STORM WATER MANAGEMENT MODEL

VI-1 INTRODUCTION

The Environmental Protection Agency Storm Water Management Model (38) hereafter referred to as SWMM, has been applied to the 335.8 hectare (829.3 acre) drainage area in Racine, Wisconsin which contributes to the two combined sewer overflow plants described in Section IV. The application, modification and results of the SWMM for this area will be discussed in the following pages.

SWMM Description

The SWMM is a packaged computer model available from the EPA which predicts for a given rainfall event the quantity and quality of storm water runoff and the resulting combined sewer overflow plus the effects of this overflow on the receiving body of water. The user of the SWMM supplies the rainfall intensity, the physical description of the land, the conveyance mechanisms, any storage-treatment systems within the drainage area and the receiving body of water. The format for this input data is described in Volume III of the SWMM which is the User's Manual (38). The output of the SWMM is in the form of hydrographs and pollutographs; that is, flow versus time and quality constituents (5-day BOD, total suspended solids, total coliforms) versus time. The Receiving block also provides velocity, stage, and dissolved oxygen concentration versus time. This form of the output allows for a time step analysis of the data as opposed to the overall effects such as total flow discharged per storm.

The SWMM program consists of over 10,000 Fortran statements which are divided into five subprograms or blocks: Executive, Runoff, Transport, Storage and Receive. The Executive block is used for control and does no computation as such. The Runoff block computes the quantity and quality of the storm water runoff for each subarea. This runoff is then applied to the various inlets of the main sewer system. The Transport block routes the runoff and dry weather flow through the conveyance system and then produces hydrographs and pollutographs at any selected point within the drainage area. The Storage block modifies the output of the Transport block according to the user's selection of various storage and/or treatment facilities provided in the program. Thus, dissolved air flotation or microstrainers might be selected as one treatment option. The Receiving block uses the output of Transport or Storage and computes the effect of the discharge on the receiving river, lake, or bay. Figure 60 shows the interrelationship and the general type of input data for each block.

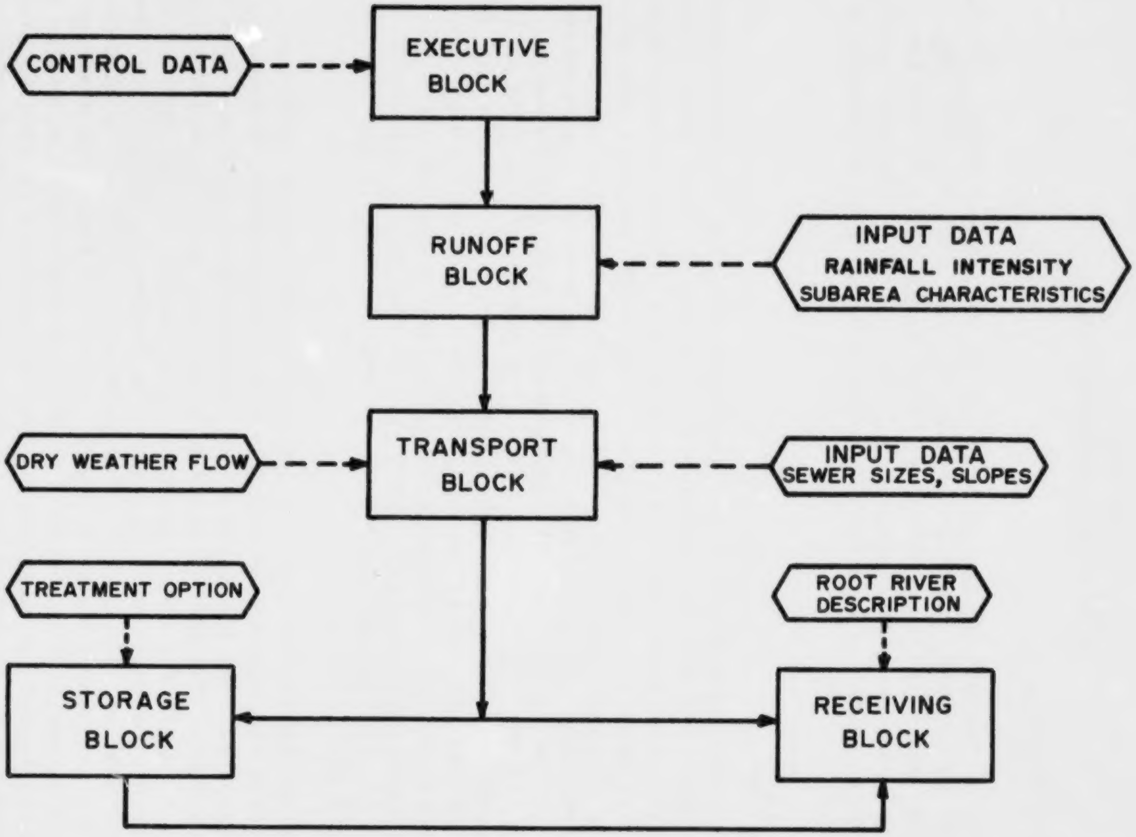


Figure 60. SWMM block diagram.

Data Requirements For Racine

The SWMM requires a large amount of input data to describe the drainage area and the receiving body of water. The Runoff block uses the characteristics of the drainage area such as subarea land use, surface slope, and percent imperviousness along with the allocation of rainfall intensities to each subarea to determine the amount of runoff to the sewerage system. The Transport block requires the description of the sewerage system and the dry weather flow for each subarea. Thus, the size, slope, roughness coefficient and upstream element are required for each sewer and gutter. The dry weather flow of the area is determined by the population, number of households and major industrial flows of each subarea. The Storage block requires the description of a storage and/or treatment device selected by the user to treat an overflow. In Racine, the two screening/dissolved-air flotation units are used as treatment options with no associated storage facility. The Receiving block requires a description of the flow, velocity, depth, stage and loadings for the receiving body of water. The final 10 kilometers (6 miles) of the Root River are used as the receiving body. Section V of this report describes the Root River monitoring program.

The comparison of the output of the SWMM to actual measured data is an important part of this report. The two combined sewer treatment plants provide flow measuring devices and sampling points at each overflow. The large amount of data that was generated during the 45 monitored overflow events provide the basis for comparison with the SWMM output. In the following portions of this section of the report, the data used for each block will be described, the initial results will be discussed along with any problems and then the total program output will be evaluated. The final topic of this section will be the application of the SWMM to the remaining combined sewer areas of the city and the results of these discharges on the Root River.

VI-2 RUNOFF AND TRANSPORT BLOCKS

Because of the close relationship between the Runoff and Transport data, both of these blocks will be described together throughout this section.

Data Acquisition

The collection and preparation of the data used as input for these blocks began shortly after the selection of the outfall locations for the construction of the treatment units. Utilizing maps of the sewer system supplied by the Racine City Engineer's Office, it was possible to determine the boundaries of the drainage area which contributed to these overflow points. Next, the interceptor, trunk (main) and branch sewers were located in the drainage area and the total area was divided into a number of smaller areas based on the layout of the sewerage system. These areas were then examined to determine the direction that the runoff flowed and the entry point or inlet to the sewer system. The direction of flow was determined by using street corner elevations from the sewer maps. If the runoff was entirely directed to a single inlet, then the subarea remained. But if the runoff flowed to two different inlet points within the original subarea, then that subarea was further divided into two separate subareas. After the runoff patterns were determined, the land use within each common runoff area was listed. If an area was composed of residential and commercial land uses, then this area was further subdivided along these land use boundaries. The total drainage area was finally divided into 56 subareas according to the following land use patterns:

Single family residential	20 subareas	210.4 hectares	(519.9 acres)
Multi-family residential	13 subareas	33.1 hectares	(81.8 acres)
Commercial	14 subareas	40.1 hectares	(99.1 acres)
Industrial	6 subareas	32.7 hectares	(80.8 acres)
Parkland	3 subareas	19.0 hectares	(46.9 acres)

Figure 61 shows the location of these subareas and the numbers used to identify them throughout the SWMM. The percent imperviousness of each subarea was determined by use of an aerial photograph. This photograph was analyzed for the amount of pavement, roof area or other hard surfaces that caused runoff to the conveyance system. This area was then expressed as a percent of the total area. After the percent imperviousness was determined for each subarea, the sewer maps were again used to determine whether the surface runoff went directly to an inlet of the main conveyance system or was transferred by means of a gutter pipe. In most cases a gutter pipe was the primary means of drainage, with the flow eventually reaching an inlet to the main conveyance system. It must be noted at this time that subarea numbers 14, 17, 47, and 48 are assigned a contributing area of only 0.4 hectares (0.1 acre) because these areas do not contribute surface runoff to the conveyance system but only dry weather flow. Thus, these areas are assigned minimal runoff contributing area and actual population equivalents for the dry weather flow contribution.

The elements selected to represent the drainage area were now completely described. Figure 62 has been constructed showing the conveyance system within the drainage area. There are 26 gutter pipes in the drainage area ranging in length from 61 meters (200 ft) to 1,051.5 meters (3,450 ft). The total length of all gutters is 6,167.7 meters (20,235 ft). A total of 50 sewer elements, ranging in length from 3 meters (10 ft) to 1,024 meters (3360 ft) with a total length of 119,638 meters (392,515) were modeled. The diameter of these sewers ranges from 0.2 meters (0.66 ft) to 3.35 meters (11.0 ft). The slope of these sewers was determined by taking the difference in elevation between two manholes at the end of each element and dividing by the length of the sewer. The elevations were obtained from detailed sewer maps which were later found to be lacking in accuracy during spot checks of the system. A detailed listing of the slopes was not available and the present procedure was used as the only alternative. The manholes assigned to the conveyance system of the Transport block were placed wherever a change in slope or a branch in the system occurred. There are 68 manholes in the system. The other group of elements used in the conveyance system are the flow dividers. These elements are used to route a

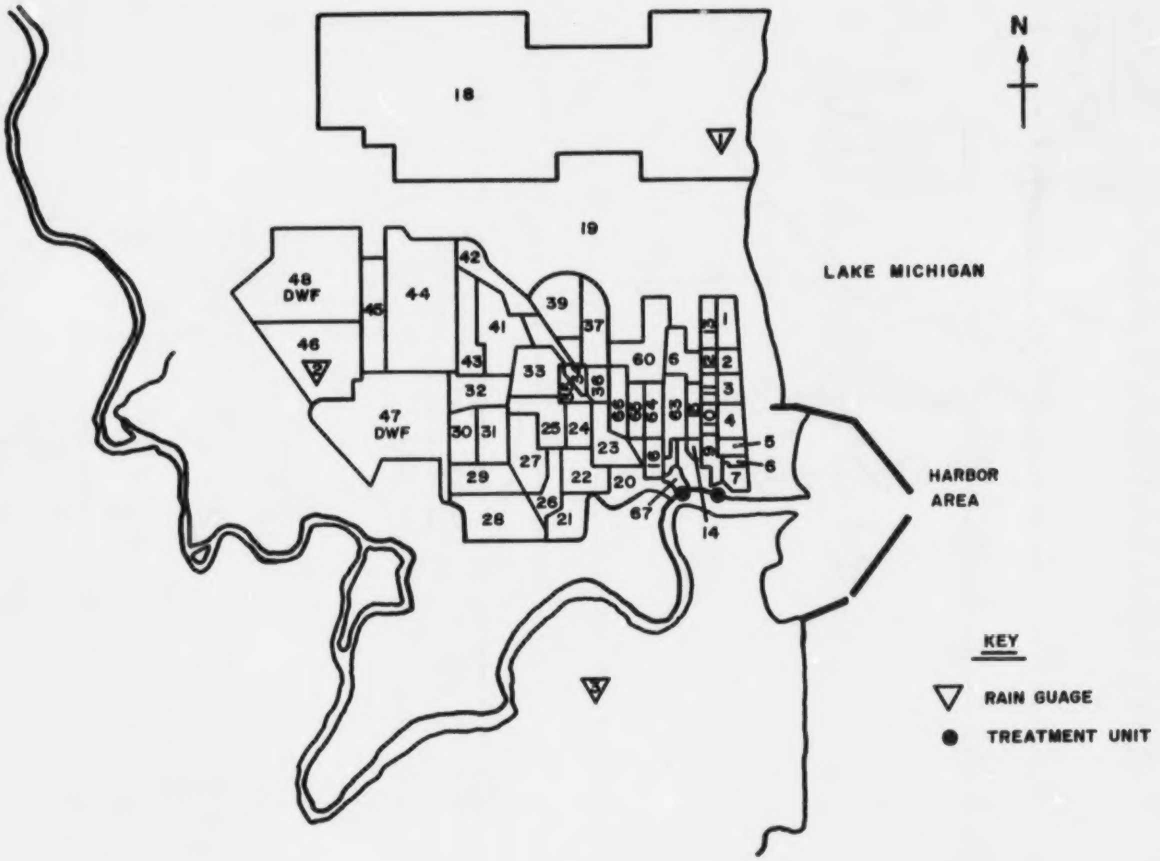


Figure 61. Subarea locations and numbers.

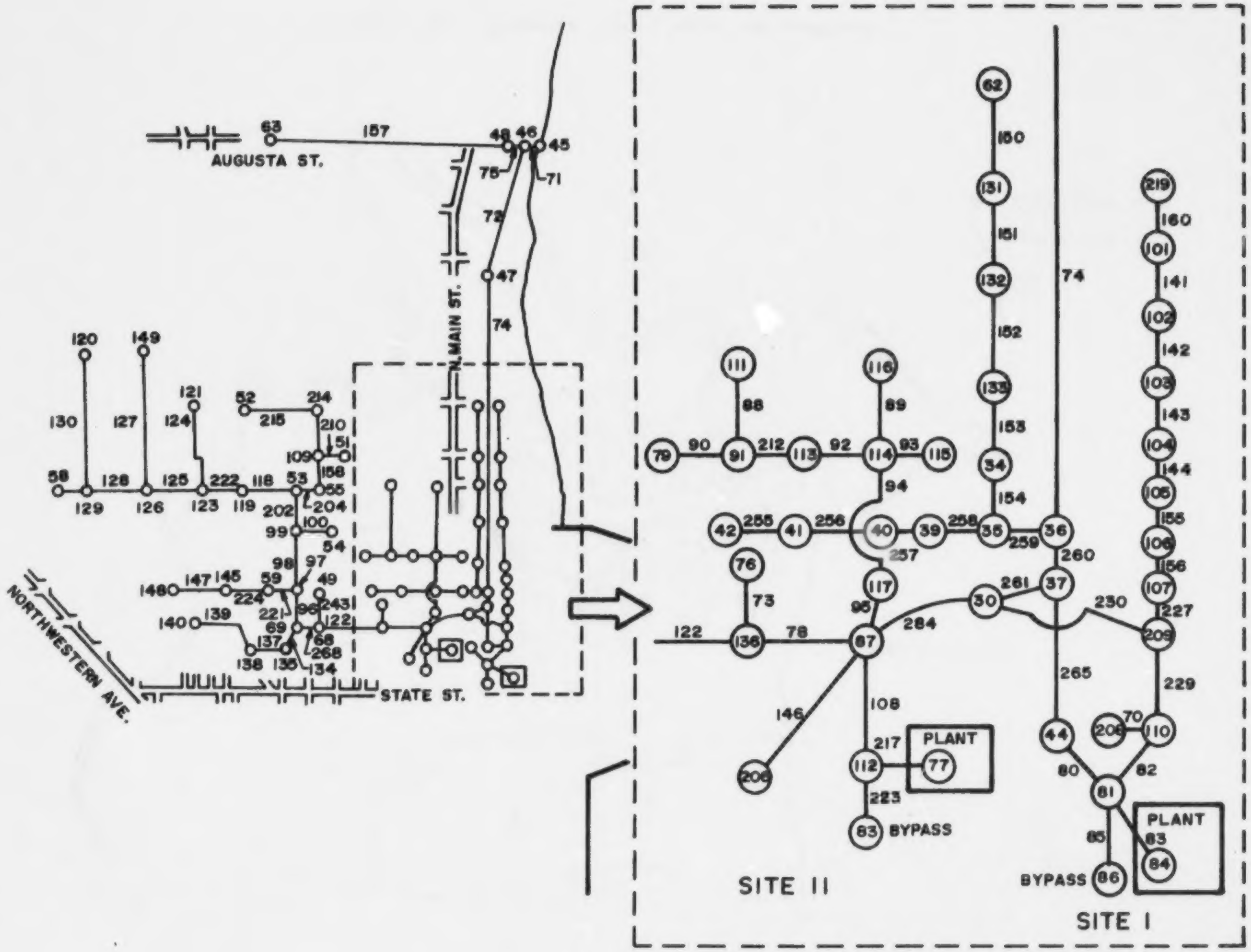


Figure 62. Transport elements.

portion of the flow from the main system to an overflow point or to divide the flow between two branching elements. The data used to describe the flow dividers are the diameter of the contributing sewers, the height of the dam or weir in these sewers, the weir constant and the number of the element into which the diverted and undiverted flow is routed. Figure 62 shows the six flow dividers in the conveyance system, numbered 81, 112, 209, 37, 87, and 46. Numbers 81 and 112 are the wetwells of the two treatment units. The flow that arrives at these wetwells is pumped to the two treatment units with any flow in excess of the plant capacity bypassed to the river. Thus, elements 81 and 112 are flow dividers with undiverted capacities of 42.1 cu m per min (24.8 cfs) and 116.8 cu m per min (68.7 cfs) respectively. Numbers 37, 209, and 87 are the major flow dividers that determine when flow is to be routed to the treatment units. During dry weather, these elements prevent the passage of dry weather flow to the treatment units. Number 46 routes the flow from the 145.8 hectare (360 acre) subarea of the drainage system to either the main sewerage system in dry weather or during wet weather when flows exceed 49.4 cu m/min (29.1 cfs) the flow is bypassed to Lake Michigan. Since the amount of flow necessary to cause bypass is relatively large, this element contributes significantly to the downstream treatment units during wet weather which receive the undiverted flow.

The Transport block requires the hourly and daily variations in the quality and quantity of the dry weather flow of the drainage area. This variation is necessary because the computations of the SWMM are dependant on the real time of occurrence of the rainfall event. The Racine Water Pollution Control Plant records provided the hourly and daily variation in the quantity of the flow which was expressed as a ratio of the mean yearly flow. The variation in the quality of this flow could not be fully determined using the treatment plant records since the dry weather samples are taken as 6 hour composites with no total coliform analysis performed. For example, during the summer of 1974, dry weather flow entering the treatment plant was sampled every hour for each day of the week during a dry weather period. The resulting 168 samples (24 samples per day for 7 days) were analyzed for TOC and the Sunday, Monday, Wednesday, and Friday samples were also analyzed for five day BOD, total suspended solids, and total coliforms. The Tuesday, Thursday, and Saturday TOC values were then used to predict the BOD variation for these days using simple correlation analysis. The suspended solids and total coliform variations were determined by averaging the other corresponding values of the week and weighing them according to the treatment plant's 6 hour composites. Thus, the hourly and daily variation for the quality of the dry weather flow was completed and could be used as input to the SWMM. The average quantity of the dry weather flow for the entire drainage area can be input to the Transport block or the SWMM can compute this value using the population estimates provided for each subarea. Initially the dry weather flow for each subarea was estimated by determining an average dry weather flow per acre for the entire city. This procedure was later felt to be inaccurate and during the fall of 1973 a portion of the dry weather flow from the drainage area was actually measured and these data were then used to determine the average dry weather flow rate. This value, along with the hourly and daily variation in the quantity and quality, provided a complete record of the dry weather flow of the drainage area.

The amount of rainfall for each overflow event was measured by three raingauges placed throughout the city as shown in Figure 61. A description of each gage and their operation is presented in Section V of this report. Each gage recorded the cumulative amount of rainfall versus time by means of an inked line on a strip chart. The values from these charts were then converted into inch-per-hour intensities at five minute intervals. Because of difficulties in reading the raingage values for certain times of the rainfall, the intensities from each gage were subject to variations in the real time of occurrence. This is important to remember when comparing the SWMM output to actual conditions. The contributing area of each raingauge was determined graphically by constructing the perpendicular bisectors of the lines joining the location of each raingauge on a map. The polygons that were formed gave an approximate outline of the contributing area. The subareas not covered by a polygon were then assigned to the closest raingage and all subareas east of Main Street were assigned to raingage I due to the difference in rainfall intensities near Lake Michigan.

Input Data

The data used in the Runoff block to describe the drainage subareas are listed in Table A1, Appendix VI-A. The first column of this table lists the subarea number from Figure 61. Note that the numbering system is 1 to 48 and 60 to 67. This was done to accomodate other modeled areas outside of this main combined sewer area. The following columns either list the number of the gutter/pipe used to convey the surface runoff to the main system or the number of the inlet to the main system that receives this runoff. The width, area, percent imperviousness, ground slope, raingage number and land use within each subarea is also listed. The input data used to describe the gutter/pipes of the conveyance system are shown in Table A2, Appendix VI-A. The number of the gutter/pipe and inlet to the main conveyance system are shown in the first two columns. The width (diameter), length and slope are also presented in this table. The placement and format of this input data for the Runoff block is listed in Table A3, Appendix VI-A.

The input data used for the Transport block includes the description of the conveyance system elements (manholes, sewers, flow dividers, gutter/pipes) and the data used to characterize the dry weather flow of the area. Table B1, Appendix VI-B lists the data used to describe the manholes of the conveyance system. The upstream elements that are listed in this table provide the SWMM with the types of branching and flow routing characteristic of the conveyance system. Thus, manhole No. 114 has 3 upstream conduits numbered 89, 93, and 92 that flow into it. Table B2 Appendix VI-B lists the input data for the 76 sewers and gutter pipes of the area. All of these elements are circular shaped having their diameters equal to their widths. Two of these elements are dummy sewers that are used to connect the treatment units to the bypass channel of the wetwell. These elements were added to allow each treatment unit to be a single input to the receiving water rather than the effluent from the treatment plant plus the bypass being added separately at each site. This would mean that the receiving water would need 4 inputs in less than 305 meters (1000 ft).

The characteristics of the dry weather flow of the drainage area were obtained from population estimates, and the Racine Water Pollution Control Plant records. The hourly variations in the quality of this flow were determined by a week of actual sampling. The average yearly flow, BOD and suspended solids concentration were then compared to the daily averages and hourly values obtained from the dry weather sampling.

A ratio of the daily average to the yearly average produces the daily correction factor and the ratio of the hourly average for the week to the yearly average provides the hourly variation for flow and quality constituents. Table B3, Appendix VI-B lists these ratios as they are used as input to the SWMM. Table B4, Appendix VI-B provides the population densities and actual populations for each subarea that has a residential land use. Note that subareas 47 and 48 are the separated areas that only contribute dry weather flow to the conveyance system. In order to provide an accurate population equivalent for these areas, the population density used was 405 persons per hectare (9999 per acre) since the contributing area is only .04 hectare (0.1 acre). The other method of accounting for the dry weather flow in the conveyance system was to assign process flows to those points in the area where the dry weather flow from non-modeled areas enters the system. Thus, at manholes 46 and 47 process flows are added to account for the contribution of the areas north of subarea 18 and the contribution from another separated area, No. 19. These process flows contain the yearly average dry weather BOD and suspended solids concentrations. The magnitude of these flows was determined by use of data provided in an infiltration study performed for the City of Racine (personal communication from Donohue and Associates, Consulting Engineers, Sheboygan, Wisconsin). This study provided the flows for each of these areas as they entered the main interceptor in the conveyance system. During November 1973 a portion of the sewer system north of Site I was increased in size to prevent problems with surcharging along this line. Sewer number 230 in Figure 62 shows the repaired sewer location. While this sewer maintenance was undertaken, the dry weather flow that normally passes through this sewer was diverted to Site I and treated. The flow measurements taken during this period provided the basis for modifying the computed dry weather flows to fit these measured values. The final results of these modifications are listed in the Transport block input data shown in Table B5, Appendix VI-B.

The Runoff and Transport blocks were now operational and the output of these blocks was investigated to determine where calibrations could be made if needed. But the application of the SWMM in this project was to an existing area which provided few such possibilities. The only real calibration that occurred was with the computed dry weather flow. Initial estimates of this flow for the conveyance system of the area were approximately 6.8 cu m/min (4.0 cfs) while the measured value was 3.6 cu m/min (2.1 cfs). By adjusting the total discharge area average sewage flow (ADWF) of the SWMM to be measured value, the computed dry weather flow rate became closer to actual conditions. Both of these blocks were now ready to be used in comparison with actual rainfall events and measured values.

Before the initial results with the SWMM are presented, the problems with the flow measuring devices and data acquisition will be discussed. The two treatment units discussed in a previous section provide a means for monitoring the overflows. The wetwells of each unit (element numbers 81 and 112) have pumps that remove the flow at a constant rate equal to the plant capacity. All flow below this value is removed to a treatment unit and any flow arriving in excess of this value is bypassed to the Root River. The plant flow is measured by a Parshall flume which is downstream of the plant pumps. The bypass flow is measured by a bubbler tube placed at the bypass weir. Both of the flow measuring devices record the flow in gallons per minute on a circular time chart. The plant flow values are obtained from a bulb that rides on the water surface in the Parshall flume. Because of variations in the water surface through the flume, a range of values for the plant flow readings at Site I will be presented. Thus, a typical flow range may be 5.1 to 7.0 cu m/min (3.0 to 4.1 cfs). The flow measurements at Site II were found to be faulty and appropriate correction factors had to be applied. Because of these corrections only a single line will be presented for the Site II plant flow. The bypass recorder at Site I also required a correction factor. Both of these factors are discussed in Section IV of this report. Throughout the following comparisons, the arriving flow at each wetwell will be compared to the computed flow. This means that the plant flow plus the bypass flow are added together for each unit of time and compared with the SWMM output for the arriving flow. This procedure allows better comparisons since once the arriving flow exceeds plant capacity, the plant flow remains constant at 42.1 cu m/min (24.8 cfs) for Site I and 116.8 cu m/min (68.7 cfs) for Site II. The majority of the comparisons will be done for Site I since this site provides accurate plant flow measurements and bypass records, the simplest flow divider situations and least amount of mechanical problems that caused variations in the monitoring of the overflow. Site II requires a correction factor for plant flow for all runs and for the first year of operation, no bypass record was obtained. This site also has two gates in the sewers north of the plant that open and close during various runs to either store some of the arriving flow in line or to use this treatment site at or near capacity when flows in the interceptor are low. Thus, in order to model these different physical situations would require that certain storms be run in sections according to the configuration of these gates. The operation of these gates is explained in Section IV under the Design and Construction subsection When complete data is available, both sites will be used for comparison.

The quality data used for comparison at both sites is obtained from seven discretely sampled overflows. All other runs have composite samples that are proportioned according to flow. These values will be presented as a basis for comparison to the SWMM output with the discrete data providing a more accurate and detailed comparison.

Initial Results

The comparison of the SWMM output to the measured arriving flows at the treatment units began with run No. 12 which occured on July 20, 1973.

This was the first run which provided accurate raingage data, good flow measurements at Site I and a significant amount of rain, 3.3 centimeters (1.3 inches) in 3 hours. The rainfall intensities along with the other input data related to this run are shown in Table E1, Appendix VI-E. Only Site I is used in this comparison because of mechanical problems with Site II. Because of the large amount of rain, the plant capacity was exceeded from 2:00 to 4:30 pm. The resulting bypass and plant flow record and the computed values are shown in Table 72. Figure 63 represents the graphical comparison of the arriving (plant flow plus bypass) flow. As this figure indicates, the computed flow lags behind the measured at the start of the run and terminates before the measured. The long duration of the measured flow is thought to be caused by infiltration since it does not occur for all overflow events. Outside of the early termination of the computed flow, the flow comparison was relatively close. The quality of the arriving flow was determined by a composite sample taken over the duration of the overflow. Using this value to determine the total kilograms (pounds) arriving for BOD and suspended solids, and determining the same for the computed, a rough comparison can be made. Thus, the following values resulted;

Kilograms (lb) BOD arriving		Kilograms (lb) suspended solids arriving	
computed	actual	computed	actual
1869 (4117)	1703 (3751)	2988 (6581)	3691 (8130)

Little, if any comparison can be made between these values at this time.

The next overflow event used for comparison is Run No. 16 which occurred on September 24, 1973. Only Site I data are available because of mechanical difficulties at Site II. Total rainfall for this run was 1.52 centimeters (0.6 inches). The rainfall data used for this storm is shown in Table E2, Appendix VI-E. A graphical comparison shows that the computed flow lags behind the measured at the start of the overflow but then passes above the measured and remains there for the duration of the overflow (see Fig. 64). This run was below plant capacity so that no bypass flow was recorded. Table 73 lists the measured and computed flows. In Run No. 12, the majority of the flow was bypassed and the computed flow was less than the measured. In Run No. 16 with no bypass, the computed flow is greater than the measured for a majority of the overflow. The quality comparison for this run is based on the kilogram (pounds) of BOD and suspended solids arriving at the treatment unit. The results are:

Kilograms (lb) BOD arriving		Kilograms (lb) suspended solids arriving	
computed	actual	computed	actual
174 (383)	445 (980)	1360 (2996)	1026 (2260)

TABLE 72. ARRIVING FLOW, SITE I
Run No. 12

Time	Arriving flow,				Computed flow,	
	cu m/min		cfs			
hr	min.	max.	min.	max.	cu m/min	cfs
1355	0.0	0.0	0.0	0.0	0.0	0.0
	41.7	45.4	24.5	26.7	3.2	1.9
1400	41.7	45.4	24.5	26.7	24.0	14.1
	94.4	98.1	55.5	57.7	21.0	12.3
	102.2	105.9	60.1	62.3	38.6	22.7
1430	106.0	110.7	62.3	64.5	68.7	40.4
	106.0	110.7	62.3	64.5	81.4	47.9
	106.0	110.7	62.3	64.5	97.2	57.2
1500	109.7	113.4	64.5	66.7	82.8	48.7
	107.8	111.5	63.4	65.6	86.0	50.6
	109.7	113.4	64.5	66.7	95.2	56.0
1530	104.1	107.8	61.2	63.4	92.5	54.4
	100.4	104.1	59.0	61.2	90.6	53.3
	101.0	94.7	53.5	55.7	80.8	47.5
1600	101.0	94.7	53.5	55.7	76.5	45.0
	83.4	87.1	49.0	51.2	66.3	39.0
	75.7	79.4	44.5	46.7	61.7	36.3
1630	64.3	68.0	37.8	40.0	63.1	37.1
	41.7	45.4	24.5	26.7	49.0	28.8
	41.7	45.4	24.5	26.7	34.0	20.0
1700	34.9	40.1	20.5	23.6	23.1	13.6
	34.0	40.8	20.0	24.0	14.5	8.5
	28.1	39.9	16.5	20.5	7.3	4.3
1730	25.3	32.5	14.9	19.1	2.0	1.2
	24.5	28.4	14.4	16.7	0.0	0.0
	23.5	27.2	13.8	16.0		
1800	20.4	24.1	12.0	14.2		
	21.3	25.2	12.5	14.8		
	22.8	26.5	13.4	15.6		
1830	17.5	20.2	10.3	11.9		
	15.8	19.0	9.3	11.2		
	13.6	18.0	8.0	10.6		
1900	13.3	16.3	7.8	9.6		
	13.6	16.0	8.0	9.4		
	15.8	18.4	9.3	10.8		
1930	18.2	21.6	10.7	12.7		

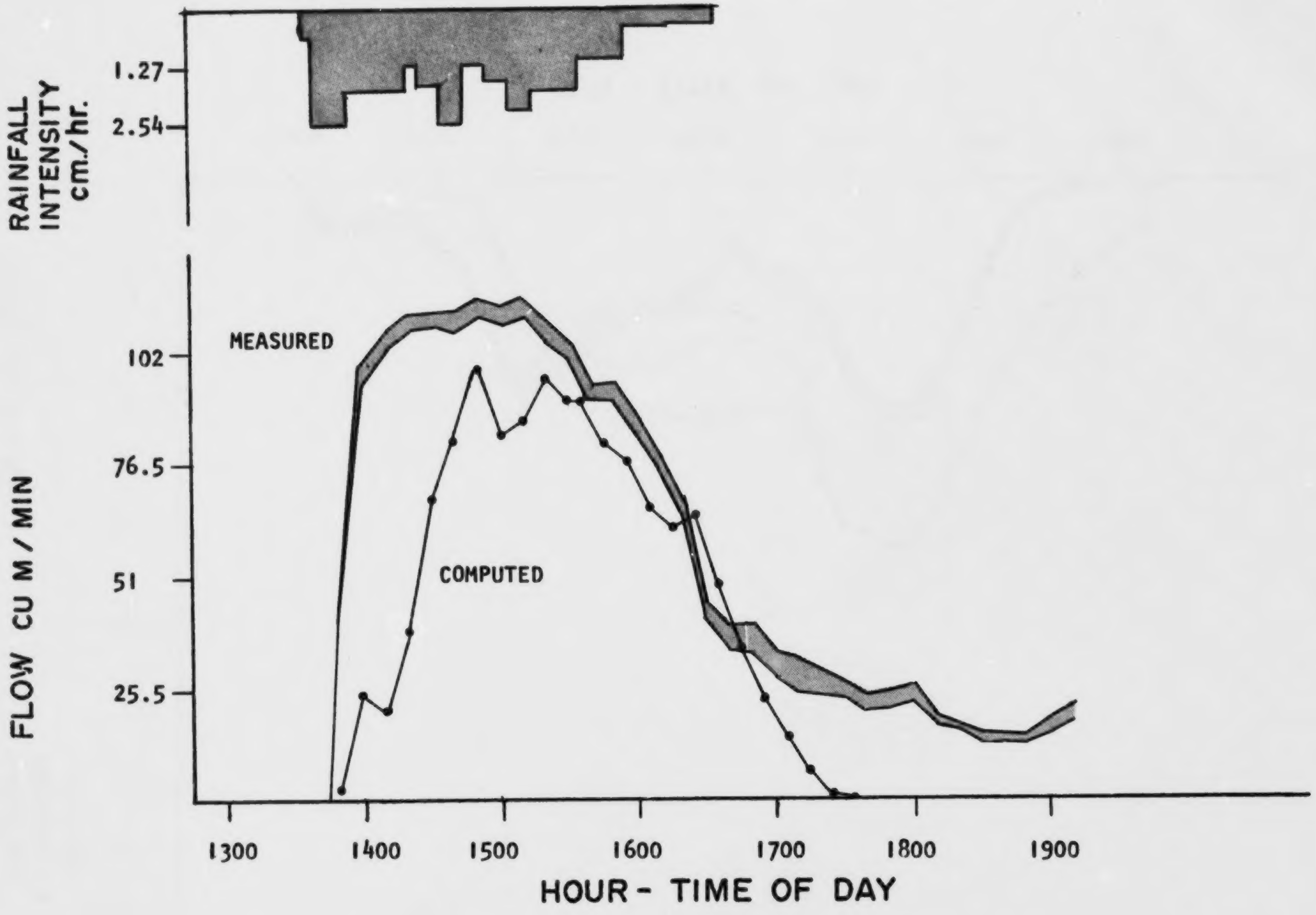


Figure 63. Run Number 12 arriving flow Site 1.

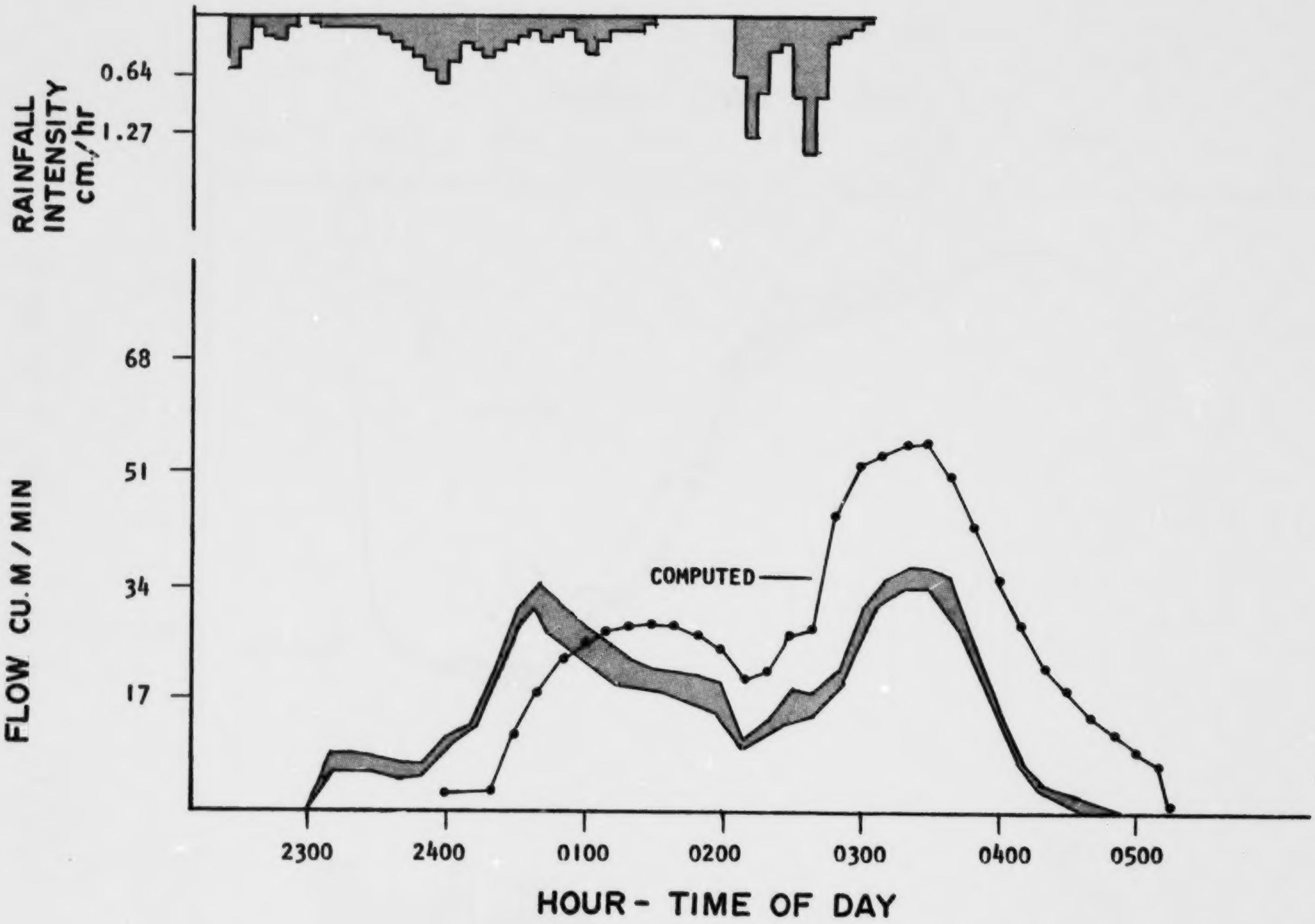


Figure 64. Run No. 16 arriving flow Site I.

TABLE 73. ARRIVING FLOW, SITE I

Run No. 16

Time	Arriving flow,				Computed flow,	
	cu m/min		cfs		cu m/min	cfs
hours	min.	max.	min.	max.		
2300	0.0	0.0	0.0	0.0	0.0	0.0
	8.3	13.9	4.9	8.2	0.0	0.0
	8.2	13.6	4.8	8.0	0.0	0.0
2330	8.2	13.3	4.8	7.8	0.0	0.0
	7.8	13.3	4.6	7.8	0.4	0.2
	9.9	15.3	5.8	9.0	0.0	0.0
2400	13.6	18.9	8.0	11.1	2.5	1.5
	19.6	22.4	11.5	13.2	2.7	1.6
	26.4	28.9	15.5	17.0	3.4	2.0
0030	32.8	37.7	19.3	22.2	11.5	6.8
	25.5	29.2	15.0	17.2	17.7	10.4
	22.4	28.6	13.2	16.8	22.6	13.3
0100	25.7	31.0	15.1	18.2	25.2	14.8
	23.8	28.7	14.0	16.9	26.9	15.8
	21.3	28.7	12.5	16.9	27.5	16.2
0130	20.4	25.7	12.0	15.1	27.7	16.3
	20.7	25.8	12.2	15.2	27.7	16.3
	18.4	23.0	10.8	13.5	26.5	15.6
0200	12.1	17.3	7.1	10.2	23.8	14.0
	16.8	21.3	9.9	12.5	20.2	11.9
	18.7	23.1	11.0	13.6	20.7	12.2
0230	18.2	22.6	10.7	13.3	26.0	15.3
	27.0	33.2	15.9	19.5	27.4	16.1
	32.6	39.1	19.2	23.0	44.7	26.3
0300	34.9	39.8	20.5	23.4	52.2	30.7
	34.9	39.8	20.5	23.4	53.9	31.7
	34.9	39.8	20.5	23.4	55.8	32.8
0330	34.9	39.8	20.5	23.4	55.8	32.8
	18.7	23.8	11.0	14.0	50.7	29.8
	10.5	14.0	6.2	8.2	42.5	25.0
0400	11.2	15.1	6.6	8.9	34.5	20.3
	6.5	8.3	3.8	4.9	27.7	16.3
	2.7	4.8	1.6	2.8	22.3	13.1
0430	1.7	3.4	1.0	2.0	17.9	10.5
	0.0	0.0	0.0	0.0	14.3	8.4
					11.4	6.7
0500					8.8	5.2
					6.8	4.0
					5.1	3.0
0530					3.6	2.1
					2.2	1.3
0550					1.0	0.6
					0.0	0.0

These results are the opposite of run Number 12 in that the computed BOD is less than the measured and the computed suspended solids is greater than the actual. In order to effectively compare the quality predictive portion of the SWMM, it was decided at this time that a run was needed with discrete sampling of the arriving flow at the treatment sites. This run would also require good raingage and flow measurement data in order to effectively compare the computed and actual quality constituents. Run No. 21 was selected and it is discussed in the portion of this report which discusses the discretely sampled runs (see Sec. VI-5; TOTAL PROGRAM EVALUATION).

VI-3 STORAGE BLOCK

The Storage block of the SWMM provides the capabilities for storing all or part of the flow in selected elements and/or treating this flow using one of several treatment options provided. For purposes of this project, only the treatment portion of this block was used since there are no storage facilities in the drainage area in Racine. The Storage block provides the option of either designing a treatment facility to the maximum arriving flow or using a design flow rate provided as input to size various processes within the facility. This latter option was used throughout this application since the screening/dissolved-air flotation units used for treatment have already been constructed.

Data Acquisition

A complete description of the two treatment units is presented in Section IV-2 of this report. The data used to describe the characteristics of these units were obtained from the as-built specifications. The treatment options which were selected from the User's Manual correspond to the existing units with the following components:

Bar racks
Inlet pumping
Fine screens and dissolved-air flotation
No secondary treatment
No effluent screens
No outlet pumping
No chlorine contact tank (chlorine added in #3).

Input Data

The Storage block receives the routed flow from the Transport block at element numbers 81 (Site I) and 112 (Site II) of Figure 62. The block is run separately for each element with the design flow rate of the treatment units provided in the input data. The pump head for the incoming flow to each site is 6.09 m (20 ft). The dissolved air flotation units are provided with chemical addition (including chlorine) and a 20% recirculation rate with a 2.6 m (8.5 ft) depth of the flotation tanks. The design overflow rate for Site I is 209.3 cubic meters per day per square meter (5131 gallons per day per square foot) for Site I and 226.6 cu m/day/m^2 (5555 gpd/ft^2) for Site II. This data was then input to the SWMM according to the formats specified in the User's Manual. Table CI, Appendix VI-C lists this data in final form for Site I and II.

Problems

After the input data for both sites were prepared, they were submitted with the needed Runoff and Transport data to the SWMM. The first few runs were used to debug the data from errors such as undefined element numbers in Transport which provide data to the Storage blocks. After the correction of these errors, further runs were needed to determine why this block continued to terminate before completion along with large amounts of asterisk and negative numbers in the printout of the inlet hydrographs and pollutographs. The input data were ruled out as the cause of this error since different runs caused the same errors. The program listing was then analyzed and the error was found to be caused by the incorrect transfer of the hydrographs and pollutographs from the Transport block to the Storage block. Thus, the BOD output of an element from Transport was used as the suspended solids input to Storage. This error was corrected by modifying about six statements in the program listing. At this time Version II of the SWMM was obtained which also contained the corrections for these errors. The new version was compatible with the original that is, it accepts data freely and has fewer possibilities of underflow or zero divide errors. It retains all of its original features, as well as an urban erosion capability, a hydraulic design section, the capability of taking two separate drainage areas and combining them into a single data set, and the addition of new treatment options in the Storage block. When the input data were submitted to this updated version, the program ran to completion without error.

Initial Results and Modifications

The Storage block was used for various runs of the SWMM to determine how this block sized the treatment units to each overflow event. At this time it was decided that the results of this block would not be acceptable because the computed treatment modules were not "sized" the same as the existing units. For example, the treatment modules of the SWMM utilized screen areas, submerged areas of screens, number of screens and design flow rates that were different than the actual conditions. These parameters were important in determining the total removals for each unit. In order to compare the computed to the actual results, it was decided to modify these parameters in the block so that the computations of this block fit the existing conditions. These changes concerned the capacity of the treatment units which are used for calculating the size and number of the bar screens and fine screens.

As was mentioned earlier, the Storage block has been developed mainly for design purposes. It determines the design flow QDESYN based on the module size of the unit, QMOD, where QMOD is determined as being as small as possible but greater than QDESYN. In order to save the actual QDESYN for further computations, the statement

```
80 QDESYN = 1.547 * QMOD(K)     TRTD 244
```

of the Storage block was changed to:

```
    80 CONTINUE
```

In this way the module size and the actual QDESYN will be utilized.

The statements used to calculate the number and size of the bar screens were:

```
    NUMBER OF
    BAR SCREENS = NSCRN = QDESYN/240

    IF NSCRN IS LESS THAN 2, LET NSCRN EQUAL 2
```

Thus, 2 screens were the minimum possible. The capacity per screen in cubic feet per second was calculated from:

```
    CAPACITY PER
    SCREEN        = SCRAP = QDESYN/NSCRN
```

The submerged area in square feet of each screen was then

```
    SUBMERGED
    AREA       = SUAREA = SCRAP/3.0
```

The face area of the screen = 1.4 * SUAREA

These statements are found in subroutine TRTDAT, statement numbers 329 through 333. The changes that were made to these statements to provide the input of existing data were:

```
    1200 READ (5,801) NSCRN, SUAREA, FAREAB

         CONTINUE
         SCRAP = QDESYN/NSCRN
         CONTINUE
         CONTINUE
```

A new data card was now placed immediately after the card group 5 which requires the design flow of the treatment unit. The new card in the data deck has the following format:

FORMAT	COLUMNS	DESCRIPTION	NAME
I10	1-10	No. of bar screens	NSCRN
2F10.0	11-20	Submerged area	SUAREA
	21-30	Face area of screens	FAREAB

The fine screens that precede the dissolved air flotation units are also sized from QDESYN. In order to include the actual screen area (SCREEN), the statement

```
3400 SCREEN = QDESYN * 449/50     TRTD 420
```

was deleted and replaced by:

```
3400 READ (5,522) SCREEN
```

An input card was then placed immediately following the level 3 treatment cards as follows.

FORMAT	COLUMNS	DESCRIPTION	NAME
F10.2	1-10	SCREEN AREA - FT^2	SCREEN

Figure 65 presents the printout of the Storage block before and after these corrections. Note the differences in the number of bar screens, submerged area and face area of the bar screens and the total fine screen area in level 3.

The Storage block was now completely operational with data defining both treatment units used in the calculations of this block. The only other change to this block was to "clean up" the printout of the "Performance Per Time Step" section of the output. Here the values associated with the listings of concentrations for certain treatment levels were either extremely large or negative when they should have been zero. Thus, when the arriving flows or overflows are zero or approaching zero (.001), the BOD or suspended solids concentrations were negative or extremely large. Figure 66 shows a typical printout of these values. Note that when the arriving flow (ARR) or the overflow (OVF) is zero, the BOD or suspended solids leaving or removed from the treatment level are very large or negative. These values do not affect any computations since the flows are zero but they do clutter the printout with unnecessary and incorrect data. These errors were caused by very small flows being used in the denominator of a calculation and the results were meaningless. To correct this procedure, the program listing was modified so that if the arriving flow or overflow was less than .01 cubic feet per second, this flow was set equal to zero. Subroutine TREA contains the calculations of these values. Table C2, Appendix VI C lists these changes and their location within the Storage block. To implement these changes, IF statements were added and other variables were set equal to zero. The results of these changes are shown in Figure 67.

The Storage block output was now ready to be compared to real data. Run Numbers 12 and 16 were used to debug and modify the data since only composites of the effluent samples were taken during the operation. Run number 21 used discrete sampling of the effluent from treatment Site 1 and was therefore used for the first comparison. The results are discussed in a later portion of this report.

VI-4 RECEIVE BLOCK

The Root River which flows through the City of Racine was used for the application of the Receive block. This block consists of two major sections

```
SPECIFIED TREATMENT CAPACITY USED.          (Before modification)
    DESIGN FLOWRATE = 60.70 CFS.
    TREATMENT SYSTEM INCLUDES MODULE UNITS
        DESIGN FLOW IS THEREFORE INCREASED TO NEXT LARGEST MODULE SIZE
        ADJUSTED DESIGN FLOWRATE = 68.70 CFS = 50,000 MGD.
        (KNOD = 8)

    NO STORAGE FROM A SEPARATE STORAGE MODEL IS ASSOCIATED WITH THIS TREAT-
    MENT MODEL

    PRELIMINARY TREATMENT BY MECHANICALLY CLEANED BAR RACKS (LEVEL 1)
        NUMBER OF SCREENS = 2
        CAPACITY PER SCREEN = 34.35 CFS
        SUBMERGED AREA = 11.43 SQ. FT  (PERPENDICULAR TO THE FLOW)
        FACE AREA OF BARS = 16.03 SQ  FT

    INFLOW BY INLET PUMPING (LEVEL 2)
        PUMPED HEAD = 20.00 FT  WATER

    TREATMENT BY DISSOLVED AIR FLOTATION (LEVEL 3)
        MODULE SIZE = 25 MGD
        NUMBER OF UNITS = 2
        TOTAL DESIGN FLOW = 50.00 MGD = 60.70 CFS
        DESIGN OVERFLOW RATE = 5555.00 CPD/SF  (5000 SUGGESTED)
        RECIRCULATION FLOW = 20.00 PERCENT (IS SUGGESTED)
        TANK DEPTH = 8.50 FEET
        TOTAL SURFACE AREA = 9593.20 SQ  FT
        CHEMICALS WILL BE ADDED
        CHLORINE WILL BE ADDED

    TREATMENT OF FINE SCREENS (AHEAD OF DISSOLVED AIR FLOTATION (LEVEL 3)
        TOTAL SCREEN AREA = 633. SQ  FT

    NO SECONDARY TREATMENT INCLUDED (LEVEL 4)

    NO EFFLUENT SCREENS (LEVEL 5)

    OUTFLOW BY GRAVITY (NO PUMPING) (LEVEL 6)

    NO CHLORINE CONTACT TANK FOR OUTFLOW (LEVEL 7)
```

Figure 65. Storage block printout before corrections.

```
SPECIFIED TREATMENT CAPACITY USED.          (After modification)

    DESIGN FLOWRATE = 60.70 CFS
    TREATMENT SYSTEM INCLUDES MODULE UNITS
        DESIGN FLOW IS THEREFORE INCREASED TO NEXT LARGEST MODULE SIZE
        ADJUSTED DESIGN FLOWRATE = 68.70 CFS = 50.00 MGD
        (KNOD = 8)

    NO STORAGE FROM A SEPARATE STORAGE MODEL IS ASSOCIATED WITH THIS TREAT-
    MENT MODEL

    PRELIMINARY TREATMENT BY MECHANICALLY CLEANED BAR RACKS (LEVEL 1)
        NUMBER OF SCREENS = 1
        CAPACITY PER SCREEN = 68.70 CFS
        SUBMERGED AREA = 550.00 SQ. FT. (PERPENDICULAR TO THE FLOW)
        FACE AREA OF BARS = 125.00 SQ. FT.

    INFLOW BY INLET PUMPING (LEVEL 2)
        PUMPED HEAD = 20.00 FT. WATER

    TREATMENT BY DISSOLVED AIR FLOTATION (LEVEL 3)
        MODULE SIZE = 25 MGD
        NUMBER OF UNITS = 2
        TOTAL DESIGN FLOW = 50.00 MGD = 68.70 CFS
        DESIGN OVERFLOW RATE = 5555.00 CPD/SF.  (5000 SUGGESTED)
        RECIRCULATION FLOW = 20.00 PERCENT (IS SUGGESTED)
        TANK DEPTH = 8.50 FT.
        TOTAL SURFACE AREA = 9593.20 SQ. FT.
        CHEMICALS WILL BE ADDED
        CHLORINE WILL BE ADDED

    TREATMENT BY FINE SCREENS (AHEAD OF DISSOLVED AIR FLOTATION) (LEVEL 3)
        TOTAL SCREEN AREA = 550. SQ. FT.

    NO SECONDARY TREATMENT INCLUDED (LEVEL 4)

    NO EFFLUENT SCREENS (LEVEL 5)

    OUTFLOW BY GRAVITY (NP PUMPING) (LEVEL 6)

    NO CHLORINE CONTACT TANK FOR OUTFLOW (LEVEL 7)
```

Figure 65 (continued). Storage block printout after corrections,

```
PERFORMANCE PER TIME STEP
NOTE:  No BOD or SS are removed in levels 2, 5, & 6, regardless of the options selected
       No SS removals in level 7 (chlorine contact tank)
       Level 1 & 5 removals (at bar racks and effluent screens) are reported in summary only


Time         -------------Inflows-------------         Fine scr + DAF     ---Bypass level 4---
             Water    BOD     SS   Coliforms          Total   BOD    SS   Total   BOD    SS
Hr:min        cfs    mg/l   mg/l  mpn/100 ml           cfs   mg/l  mg/l    cfs   mg/l  mg/l

12:35 ARR    0.00     0.      0.    0.0         OUT  0.00     -0.    -1.  0.00    -0.   -1.
      OVF    0.0      0.      0.    0.0         REM  0.00     -9.  -253.  0.0      0.    0.

12:40 ARR    0.00     0.      0.    0.0         OUT. 0.00     -0.    -1.  0.00    -0.   -1.
      OVF    0.0      0.      0.    0.0         REM  0.00     -9.  -253.  0.0      0.    0.

12:45 ARR    0.00     0.      0.    0.0         OUT  0.00     -0.    -1.  0.00    -0.   -1.
      OVF    0.0      0.      0.    0.0         REM  0.00     -9.  -253.  0.0      0.    0.

12:50 ARR    0.00     0.      0.    0.0         OUT  0.00     -0.    -1.  0.00    -0.   -1.
      OVF    0.0      0.      0.    0.0         REM  0.00     -9.  -253.  0.0      0.    0.

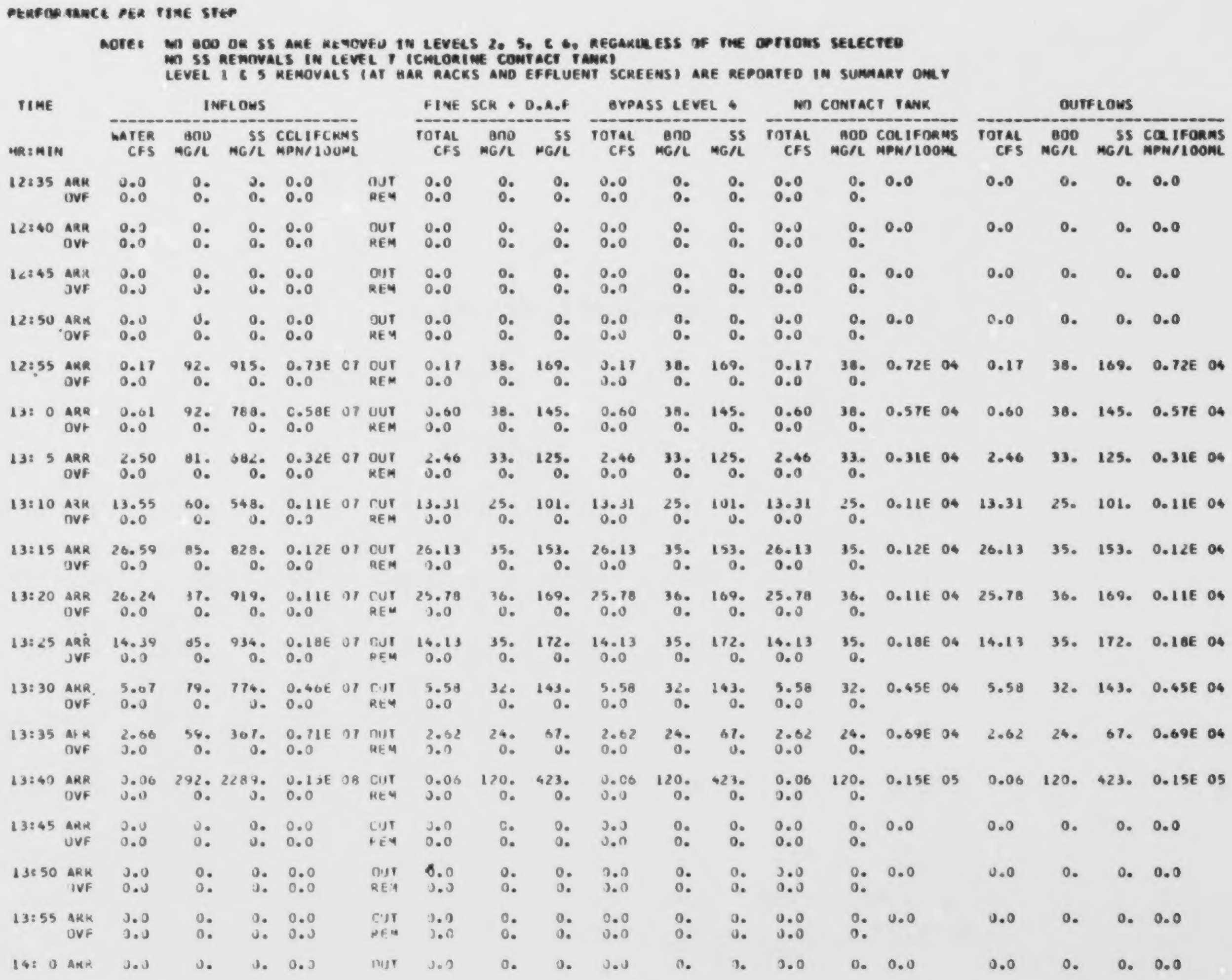
12:55 ARR    0.00     0.      0.    0.0         OUT  0.00     -0.    -1.  0.00    -0.   -1.
      OVF    0.0      0.      0.    0.0         REM  0.00     -9.  -253.  0.0      0.    0.

13:00 ARR    0.10    75.     68.    0.68F 07    OUT  0.10     31.    12.  0.10    31.   12.
      OVF    0.0      0.      0.    0.0         REM  0.00   2534.  2954.  0.0      0.    0.
```

Figure 66. Arriving flow and overflow printout values before modification.

Time		No contact tank			Outflows			
Hr:min		Total cfs	BOD mg/l	Coliforms mpn/100 ml	Total cfs	BOD mg/l	SS mg/l	Coliforms mpn/100 ml
12:35	ARR	0.00	-0.	0.00	0.00	-0.	-1.	0.0
	OVF	0.0	0.					
12:40	ARR	0.00	-0.	0.0	0.00	-0.	-1.	0.0
	OVF	0.0	0					
12:45	ARR	0.00	-0.	0.0	0.00	-0.	-1.	0.0
	OVF	0.0	0					
12:50	ARR	0.00	-0.	0.0	0.00	-0.	-1.	0.0
	OVF	0.0	0.					
12:55	ARR	0.00	-0.	0.0	0.00	-0.	-1.	0.0
	OVF	0.0	0.					
13:00	ARR	0.10	31.	0.62E 04	0.10	31.	12.	0.62E 04
	OVF	0.0	0.					

Figure 66 (continued).



```
PERFORMANCE PER TIME STEP

         NOTE:  NO BOD OR SS ARE REMOVED IN LEVELS 2, 5, & 6, REGARDLESS OF THE OPTIONS SELECTED
                NO SS REMOVALS IN LEVEL 7 (CHLORINE CONTACT TANK)
                LEVEL 1 & 5 REMOVALS (AT BAR RACKS AND EFFLUENT SCREENS) ARE REPORTED IN SUMMARY ONLY

  TIME                 INFLOWS                    FINE SCR + D.A.F    BYPASS LEVEL 4      NO CONTACT TANK            OUTFLOWS
             ---------------------------          ----------------   ----------------   ------------------   --------------------------
            WATER   BOD    SS CCLIFCRMS           TOTAL   BOD    SS   TOTAL   BOD    SS   TOTAL   BOD COLIFORMS   TOTAL   BOD    SS COLIFORMS
HR:MIN        CFS  MG/L  MG/L MPN/100ML             CFS  MG/L  MG/L     CFS  MG/L  MG/L     CFS  MG/L MPN/100ML     CFS  MG/L  MG/L MPN/100ML

12:35 ARR    0.0    0.     0.  0.0         OUT    0.0    0.    0.     0.0    0.    0.     0.0    0.  0.0          0.0    0.    0.  0.0
      OVF    0.0    0.     0.  0.0         REM    0.0    0.    0.     0.0    0.    0.     0.0    0.

12:40 ARR    0.0    0.     0.  0.0         OUT    0.0    0.    0.     0.0    0.    0.     0.0    0.  0.0          0.0    0.    0.  0.0
      OVF    0.0    0.     0.  0.0         REM    0.0    0.    0.     0.0    0.    0.     0.0    0.

12:45 ARR    0.0    0.     0.  0.0         OUT    0.0    0.    0.     0.0    0.    0.     0.0    0.  0.0          0.0    0.    0.  0.0
      OVF    0.0    0.     0.  0.0         REM    0.0    0.    0.     0.0    0.    0.     0.0    0.

12:50 ARR    0.0    0.     0.  0.0         OUT    0.0    0.    0.     0.0    0.    0.     0.0    0.  0.0          0.0    0.    0.  0.0
      OVF    0.0    0.     0.  0.0         REM    0.0    0.    0.     0.0    0.    0.     0.0    0.

12:55 ARR    0.17  92.   915.  0.73E 07    OUT    0.17  38.  169.     0.17  38.  169.     0.17  38.  0.72E 04     0.17  38.  169.  0.72E 04
      OVF    0.0    0.     0.  0.0         REM    0.0    0.    0.     0.0    0.    0.     0.0    0.

13: 0 ARR    0.61  92.   788.  0.58E 07    OUT    0.60  38.  145.     0.60  38.  145.     0.60  38.  0.57E 04     0.60  38.  145.  0.57E 04
      OVF    0.0    0.     0.  0.0         REM    0.0    0.    0.     0.0    0.    0.     0.0    0.

13: 5 ARR    2.50  81.   682.  0.32E 07    OUT    2.46  33.  125.     2.46  33.  125.     2.46  33.  0.31E 04     2.46  33.  125.  0.31E 04
      OVF    0.0    0.     0.  0.0         REM    0.0    0.    0.     0.0    0.    0.     0.0    0.

13:10 ARR   13.55  60.   548.  0.11E 07    OUT   13.31  25.  101.    13.31  25.  101.    13.31  25.  0.11E 04    13.31  25.  101.  0.11E 04
      OVF    0.0    0.     0.  0.0         REM    0.0    0.    0.     0.0    0.    0.     0.0    0.

13:15 ARR   26.59  85.   828.  0.12E 07    OUT   26.13  35.  153.    26.13  35.  153.    26.13  35.  0.12E 04    26.13  35.  153.  0.12E 04
      OVF    0.0    0.     0.  0.0         REM    0.0    0.    0.     0.0    0.    0.     0.0    0.

13:20 ARR   26.24  37.   919.  0.11E 07    OUT   25.78  36.  169.    25.78  36.  169.    25.78  36.  0.11E 04    25.78  36.  169.  0.11E 04
      OVF    0.0    0.     0.  0.0         REM    0.0    0.    0.     0.0    0.    0.     0.0    0.

13:25 ARR   14.39  85.   934.  0.18E 07    OUT   14.13  35.  172.    14.13  35.  172.    14.13  35.  0.18E 04    14.13  35.  172.  0.18E 04
      OVF    0.0    0.     0.  0.0         REM    0.0    0.    0.     0.0    0.    0.     0.0    0.

13:30 ARR    5.67  79.   774.  0.46E 07    OUT    5.58  32.  143.     5.58  32.  143.     5.58  32.  0.45E 04     5.58  32.  143.  0.45E 04
      OVF    0.0    0.     0.  0.0         REM    0.0    0.    0.     0.0    0.    0.     0.0    0.

13:35 ARR    2.66  59.   367.  0.71E 07    OUT    2.62  24.   67.     2.62  24.   67.     2.62  24.  0.69E 04     2.62  24.   67.  0.69E 04
      OVF    0.0    0.     0.  0.0         REM    0.0    0.    0.     0.0    0.    0.     0.0    0.

13:40 ARR    0.06 292.  2289.  0.15E 08    OUT    0.06 120.  423.     0.06 120.  423.     0.06 120.  0.15E 05     0.06 120.  423.  0.15E 05
      OVF    0.0    0.     0.  0.0         REM    0.0    0.    0.     0.0    0.    0.     0.0    0.

13:45 ARR    0.0    0.     0.  0.0         OUT    0.0    0.    0.     0.0    0.    0.     0.0    0.  0.0          0.0    0.    0.  0.0
      OVF    0.0    0.     0.  0.0         REM    0.0    0.    0.     0.0    0.    0.     0.0    0.

13:50 ARR    0.0    0.     0.  0.0         OUT    0.0    0.    0.     0.0    0.    0.     0.0    0.  0.0          0.0    0.    0.  0.0
      OVF    0.0    0.     0.  0.0         REM    0.0    0.    0.     0.0    0.    0.     0.0    0.

13:55 ARR    0.0    0.     0.  0.0         OUT    0.0    0.    0.     0.0    0.    0.     0.0    0.  0.0          0.0    0.    0.  0.0
      OVF    0.0    0.     0.  0.0         REM    0.0    0.    0.     0.0    0.    0.     0.0    0.

14: 0 ARR    0.0    0.     0.  0.0         OUT    0.0    0.    0.     0.0    0.    0.     0.0    0.  0.0          0.0    0.    0.  0.0
```

Figure 67. Arriving flow and overflow printout values after modifications.

which may be run together or separately. One section, designated QUANTITY, determines the hydraulics of flows for the receiving body while QUALITY determines the concentration of selected constituents at points within the modeled area. The major effort in the application of this block was with the QUANTITY section; once operational, the data obtained from Section V Root River Monitoring Studies, was used in the QUALITY portion.

Data Acquisition

The collection of data used to define the receiving body for the SWMM was initiated shortly after the Storage block was completely operational. The data needed as input included water surface elevations, depths, widths, flows, velocities, any head relationships and the loadings of selected constituents and their decay or reaeration rates. The data defining the water surface elevations and flows were obtained from the Root River Watershed Report (18) and reports from the Wisconsin Department of Natural Resources (19). From these data, the Root River from Horlick dam to the harbor entrance of Lake Michigan was selected as the modeled area. This 10 kilometer (6 mile) section of the river provides a source for upstream water quality monitoring before entering the City of Racine at Horlick dam and contains all the combined sewer overflow locations that discharge to the river.

The Receive block requires the modeled area be sectioned into a series of channels or triangles which are connected by node points or junctions. These elements are assigned numbers which are used throughout the simulation to identify the inflow points, head relationships and changes in the physical layout of the receiving body. Figure 68 presents the layout of the receiving waters that was used for the initial runs of this block. There are 16 junctions and 18 channels used to describe the area. The junctions along the river correspond to the location of the combined sewer overflow points and major storm sewer discharges to the river. The channels inside the harbor area define three triangles which are used for open bodies of water. The depths and surface elevations of the junctions were obtained by actual measurements conducted by project personnel and the Racine City Engineer's Office. The data used to describe the junctions and channels are presented in Tables 74 and 75.

The Receive block provides three options to the user to define the head relationship of the system; tidal influence, dam, or specified inflow and outflow. Therefore, a tide or dam could be applied at the harbor entrance to Lake Michigan to simulate the effects of the lake on the mouth of the river. It was decided that the input data would use a dam at Junction 16. The lake effect would be simulated by placing the elevation of the dam slightly higher than the water surface elevation of the harbor. This would tend to "hold" a small portion of the river flow in the harbor area. The total inflow that is assigned to the system was the flow that was recorded on the day in which the water surface elevations and depths were obtained. This flow was 127.5 cu m/min (75 cfs) and is applied at junction 1.

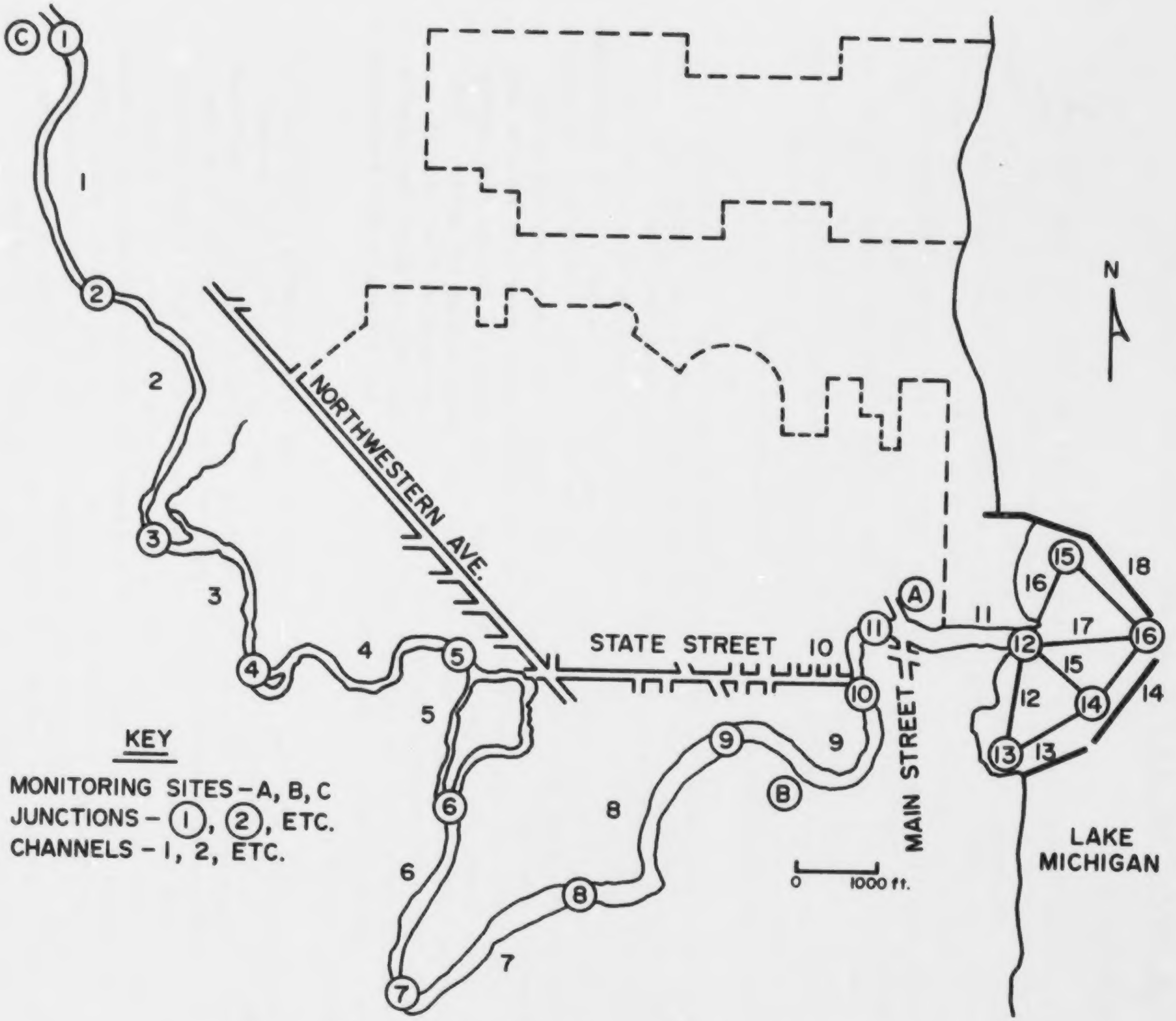


Figure 68. Receive junctions and channels (initial).

TABLE 74. RECEIVE BLOCK JUNCTIONS (INITIAL)

Junction No.	Initial head,		Depth,		Cordinates,	
	meters	feet	meters	feet	X	Y
1	187.8	616.0	186.5	612.0	0.0	12.0
2	185.6	609.0	184.7	606.0	0.8	9.5
3	183.5	602.0	182.6	599.0	1.8	6.1
4	182.0	597.0	180.4	592.0	3.2	4.4
5	180.4	592.0	179.2	588.0	5.6	4.6
6	179.2	588.0	177.4	582.0	5.3	2.9
7	177.7	583.0	176.2	578.0	5.0	0.0
8	177.7	583.0	175.6	576.0	7.0	1.8
9	177.4	582.0	174.3	572.0	8.6	3.8
10	176.8	580.0	172.2	565.0	10.7	4.6
11	176.5	579.0	169.5	556.0	10.8	5.6
12	176.4	578.8	167.6	550.0	12.6	5.5
13	176.4	578.7	174.3	572.0	12.4	3.8
14	176.3	578.4	173.7	570.0	13.5	4.5
15	176.3	578.4	173.7	570.0	13.3	6.1
16	176.2	578.0	167.0	548.0	4.2	5.5

TABLE 75. RECEIVE BLOCK CHANNELS (INITIAL)

Channel No.	Length, meters	Length, feet	Width, meters	Width, feet	Area, sq. meters	Area, sq. feet	Hydraulic Radius, meters	Hydraulic Radius, feet
1	853	2800	23	75	24.4	263	1.1	3.5
2	1524	5000	15	50	13.9	150	0.9	3.0
3	762	2500	23	75	27.9	300	1.2	4.0
4	975	3200	23	75	31.4	338	1.4	4.5
5	975	3200	23	75	34.8	375	1.5	5.0
6	853	2800	23	75	38.4	413	1.7	5.5
7	914	3000	19	63	35.1	378	1.8	6.0
8	914	3000	46	150	118.4	1275	2.6	8.5
9	975	3200	61	200	232.3	2500	3.8	12.5
10	457	1500	61	200	353.0	3800	5.8	19.0
11	457	1500	53	175	421.0	4532	7.9	25.9
12	397	1304	175	573	401.7	4324	2.3	7.5
13	410	1345	311	1019	1761.0	18956	5.7	18.6
14	522	1712	44	143	236.6	2547	5.4	17.7
15	372	1221	167	549	979.7	10546	5.9	19.2
16	488	1600	56	185	505.3	5439	9.0	29.4
17	330	1082	192	631	1125.5	12115	5.9	19.2
18	281	922	211	691	1194.8	12861	5.7	18.6

Input Data

The input data used for the initial runs of the Receive block are shown in Table DI, Appendix VI-D. This block was run separately from the other blocks with input of storm water quantity and quality from cards rather than the tape transfer from Transport. This procedure was followed until the Receive block was completely operational, then the entire SWMM was run at one time to include the outflows from the Transport or Storage block.

Problems

The data describing the receiving water were input to the SWMM with the proper job control language. The first few runs were used to debug some of the input errors that occurred because of the difficulty in interpreting the user's manual. The output of this block continued to terminate before completion of the QUANTITY section because of errors at the printout of the hydrograph inputs to the system. Numerous runs were then made with slight modifications of the input data. The output always terminated in the same location but different errors were the cause. In some cases the termination was caused by a divide by zero error or a system error. The test data from Lancaster ran without error in previous runs so the SWMM itself was not at fault. Next the program listing was consulted to determine the exact location of the errors. The SWFLOW subroutine contained the statements causing the error but the reason for the errors was not clear. At this time in the project, the overflow events of 1974 were occurring and the resulting data were used in the Runoff, Transport and Storage blocks. In addition to these runs, the Receive block data was modified in a stepwise manner to try to duplicate the test data. The changes in the input data were: increase the length of the channels by removing some of the junctions, increase the depths, use smaller time steps, remove the storm water inputs, and modify the data used to describe the dam at junction 16. All of these changes produced errors in the same location which again terminated the output. After contacting and providing the University of Florida with the input data, the following changes were recommended: remove the dam at junction 16, use the specified input-output flow and use 30 seconds for the hydraulic time step. When these changes were implemented, the output progressed past the previous errors and then terminated when the velocities in the channels exceeded 20 feet per second. The Receive block contains this "check" to insure that the computed results that follow are reasonable. The mechanics of this check are the following: if the velocity that is computed within any time step for any channel is greater than 20 feet per second, the output terminates and prints out the junction number and other hydraulic data where the error occurs. The standard fixup for this error is to increase the length of the channel so that the calculation of the velocity is taken over greater distances. Again, various modifications of the input data were undertaken to decrease the existing slope in the river and determine if the velocity decreased with this change. When this change was implemented, no significant change in the velocity resulted.

At this time various suggestions received from other SWMM users included increasing the length of the hydraulic time step or increasing Mannings coefficient by a factor of 10. Most of the suggestions were to modify the existing data to fit the SWMM. This was not the purpose of this project and further modifications of this type were dropped. The program itself was analyzed to determine if modifications could be made to allow the existing data to be input to obtain as reasonable and consistant as the output. The experience with the velocity errors indicated that the calculations of the flows for certain channels converged rapidly to zero flows and the resulting "dry channel" is then used to determine the resulting velocities which are extremely large at these small heads. The physical layout of the junctions and channels was then changed to a system consisting of only 7 junctions and 6 channels as shown in Figure 69. The data used to describe these modified junctions and channels are presented in Tables 76 and 77. The junctions of Figure 69 correspond to the major combined

TABLE 76. RECEIVE BLOCK CHANNELS (MODIFIED)

Channel No.	Length, meters	feet	Width, meters	feet	Area, sq. meters	sq. feet
1	945	3100	61	200	485	5220
2	1433	4700	53	175	236	2537
3	914	3000	61	200	91	980
4	914	3000	46	150	49	525
5	1829	6000	19	63	15	158
6	4115	13500	23	75	28	300

TABLE 77. RECEIVE BLOCK JUNCTIONS (MODIFIED)

Junction No.	Initial head, meters	feet	Depth, meters	feet	Cordinates, X	Y
1	176.2	578.0	167.0	548.0	15.0	5.5
2	176.5	579.0	169.5	556.0	10.8	5.6
3	177.4	582.0	174.3	572.0	8.6	3.8
4	177.7	583.0	176.5	576.0	7.0	1.8
5	177.7	583.0	176.5	579.0	5.0	0.0
6	180.4	592.0	179.2	588.0	5.6	4.6
7	187.8	616.0	186.5	612.0	0.0	12.0

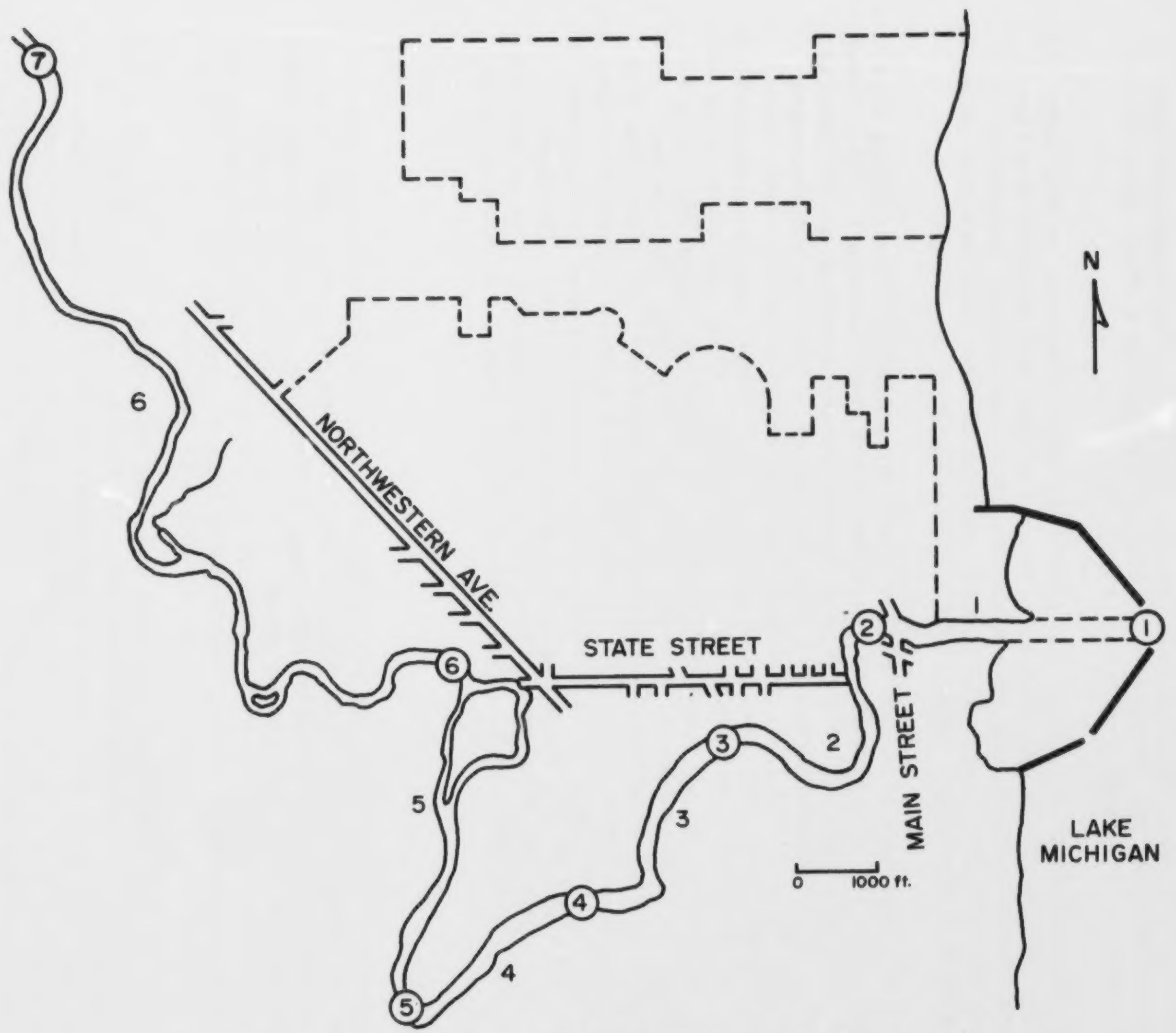


Figure 69. Receive junctions and channels (modified).

sewer overflow points of the area. The harbor area of Lake Michigan was represented as a wide channel instead of a series of triangles. To prevent confusion between the two layouts, the numbering system was changed from junction 1 as the start of the modeled area to that of the end or lowest junction. Where previously there were two junctions relatively close together, these were now combined into one junction. The downstream head relationship was now changed to a tidal effect having a small constant influence on the lower reaches of the river.

When the modified input data for this block were submitted to the SWMM, the output ran to completion without error. The head and velocity profiles along the system were not very consistent but the quantity portion of this block was now operating. The quality portion of the block was then added to simulate the entire impact of the combined sewer overflows.

The first few runs with the Receive block were obtained from running this block alone and using cards to input the stormwater characteristics. Later runs used the Runoff, Transport, and Storage blocks previous to the Receive block. This procedure required that the outfalls from the two treatment units at Sites I and II be combined into one junction in the river since two separate junctions less than 152.4 m (500 ft) apart would cause problems with the velocity determinations in the program. Thus, the wetwells at each site were connected by two dummy elements as shown in Figure 70. In order to keep the total number of Transport elements less than 150, the two elements at each treatment unit which routes the specified plant flow from each wetwell, (numbers 217, 77, 83, and 84) were dropped. Therefore, when the Storage block is run, it is applied to the wetwells at each site and any flow in excess of the design flow is bypassed as overflow. The dummy elements carry the treated outflows and overflow from each site to the new common element numbered 2, which corresponds to junction 2 in the Receive block.

Results

The entire Receive block was run using different stormwater inputs from cards or transferred by tape from storage or transport. The quantity results were extremely variable with large differences in stage between time steps for the same junction as shown in Table 78. The velocities and flows listed for each channel varied in magnitude with some being positive and others negative. Again, changes were made to the pertinent input data to rectify these problems, but no improvements resulted.

The quality determinations that were obtained from these runs were relatively constant for all parameters except DO. The initial concentrations of the four constituents at each junction were added to the input data. Thus, the initial BOD averaged 2.5 mg/l, suspended solids 30 mg/l, coliforms 500 MPN/100 ml and a DO of 7.5 mg/l. During the first day of simulation when the overflows to the river were occurring, the BOD, suspended solids and coliform concentrations did not change appreciably but the DO dropped to less than 1 mg/l. During the second day of simulation, the BOD and suspended solids concentrations started to build up in the middle of the

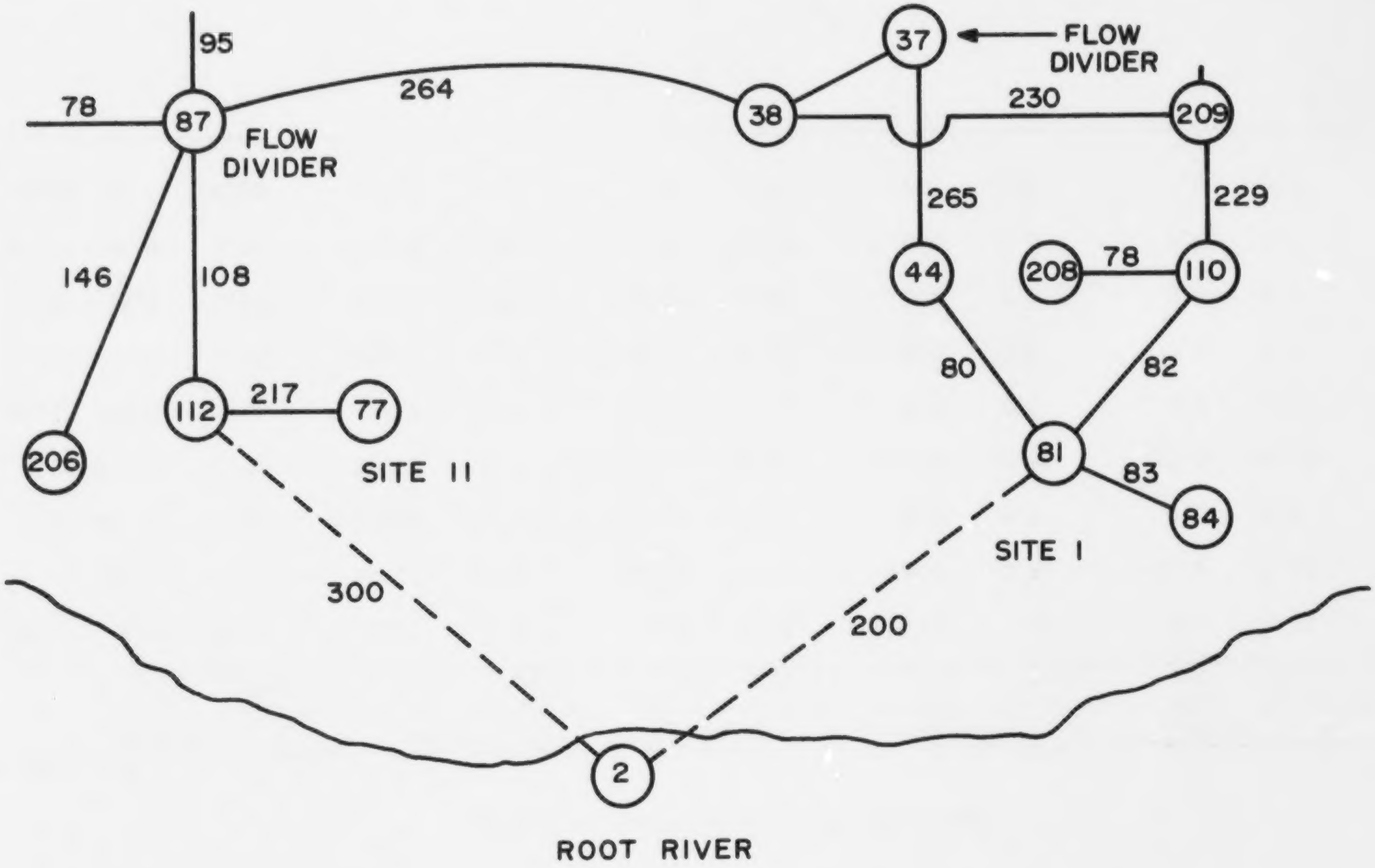


Figure 70. Treatment sites overflow modifications.

TABLE 78. RECEIVE QUANTITY (INITIAL)

Day is 1

Hour	Channel 1 2 Flow (cfs)	Channel 1 2 Velocity (fps)	Channel 2 3 Flow (cfs)	Channel 2 3 Velocity (fps)	Channel 3 4 Flow (cfs)	Channel 3 4 Velocity (fps)	Channel 4 5 Flow (cfs)	Channel 4 5 Velocity (fps)	Channel 5 6 Flow (cfs)	Channel 5 6 Velocity (fps)
0.00	0.0	0.05	0.0	0.10	0.0	0.28	0.0	1.37	0.0	1.73
3.00	552.0	0.11	492.0	0.21	269.0	0.35	-66.0	-0.57	-53.0	-1.53
6.00	-275.0	-0.05	-24910	0.10	-140.0	-0.16	0.0	0.0	0.0	0.0
9.00	-323.0	-0.06	-287.0	-0.12	-157.0	-0.19	-53.0	-0.41	-35.0	-0.97
12.00	48.0	0.01	39.0	0.01	9.0	0.01	0.0	0.0	0.0	0.0
15.00	166.0	0.03	153.0	0.06	96.0	0.12	0.0	0.0	0.0	0.0
18.00	-39.0	-0.01	-34.0	-0.01	-19.0	-0.02	0.0	0.0	0.0	0.0
21.00	-36.0	0.01	-35.0	-0.01	-21.0	-0.03	0.0	0.0	0.0	0.0
24.00	19.0	0.00	16.0	0.01	7.0	0.01	0.0	0.0	0.0	0.0

receiving body (junctions 4, 5, and 6), the coliform concentrations increased slightly throughout the system and the DO concentrations dropped to zero in all junctions. These same patterns were found for all the runs of the Receive block which used the inflows to the system which were transferred by tape from Storage or Transport or listed on cards at hour intervals for all junctions of the river. Actual river monitoring did not show these trends for any overflow events. The low DO readings that were computed were found for various decay rates, reaeration rates and initial loadings. Although the output of this block was questionable, an attempt was made to determine if the two treatment units had any effect on the computed receiving water quality. To accomplish this simulation, the receive block was run using the untreated outflows from the Transport block. The resulting water quality effects were then compared with the Receive block output when the Storage block was used to treat the overflows to the river. No difference in the quality of the river for each run was found during the three days of simulation.

VI-5 TOTAL PROGRAM EVALUATION

The following portions of this report will present the results of seven discretely sampled runs at both sites. These runs provide discrete quality data of the arriving and effluent flow from each treatment unit. In order to evaluate how well the SWMM predicts the quantity and quality of these flows, two procedures will be followed. The computed and actual hydrographs will be integrated to determine the total actual and computed flow arriving. The ratio of the actual to the computed flow will then be listed to give an indication of the SWMM's ability at predicting the total arriving flow. The manner in which these computed values follow the peaks and slopes of the measured graphs will be expressed by visually rating the goodness of fit of the two curves. If the actual hydrograph is closely paralleled by the computed, then the goodness of fit is excellent. When little correlation in the trend of the two records is found the goodness of fit is poor. Thus, even though the ratio of the total arriving flows may be 1.0, the goodness of fit may be poor. The pollutographs for each run will be compared solely on the goodness of fit of the two curves. These two methods have been selected since other methods such as correlation analysis have proved unsatisfactory due to the differences in the start and finish times between the computed and actual hydrographs and the small number of actual samples used in each analysis.

Run Number 21

The first run that provided discrete samples of the arriving flow was number 21 which occurred on November 14, 1973. The average total rainfall for the entire area was 1.94 centimeters (0.76 inches) over a duration of 400 minutes. There were 14 days prior to this run when no rain fell and 21 days for which the cumulative rainfall was less than 2.54 centimeters (1 inch). Table 79 lists the actual and computed arriving flows at Site 1 and Figure 71 presents the graphical comparison of the data. The ratio of the total measured flow arriving to the total computed flow is 0.80. The

TABLE 79. ARRIVING FLOW, SITE 1

Run No. 21

Time hours	Arriving flow, cu m/min min.		Arriving flow, cfs min.		Computed flow, cu m/min.	Computed flow, cfs
	min.	max.	min.	max.		
1650	4.9	9.9	2.8	5.8	2.0	1.2
1700	5.3	10.5	3.1	6.2	2.2	1.3
	6.1	10.5	3.6	6.2	2.4	1.4
	7.5	11.7	4.4	6.9	2.7	1.6
1730	12.1	15.8	7.1	9.3	3.6	2.1
	14.3	18.5	8.4	10.9	5.3	3.1
	15.8	22.3	9.3	13.1	6.8	4.0
1800	19.7	25.0	11.6	14.7	7.7	4.5
	24.1	30.3	14.2	17.8	8.5	5.0
	25.7	32.1	15.1	18.9	13.4	7.9
1830	24.1	32.1	14.2	18.9	23.3	13.7
	25.7	31.8	15.1	18.7	30.8	18.1
	31.8	41.8	18.7	24.6	42.0	24.7
1900	32.5	42.9	19.1	25.2	56.1	33.0
	33.8	43.2	19.9	25.4	64.8	38.1
	40.7	46.5	23.9	27.3	66.8	39.3
1930	35.8	44.2	21.1	26.0	66.8	39.8
	32.8	41.5	19.3	24.4	65.1	38.3
	31.8	39.1	18.7	23.0	59.7	35.1
2000	31.5	36.2	18.5	21.3	53.9	31.7
	29.4	33.7	17.3	19.8	48.8	28.7
	24.1	29.4	14.2	17.3	44.2	26.0
2030	23.8	28.7	14.0	16.9	40.1	23.6
	25.0	30.0	14.7	17.6	36.7	21.6
	20.4	25.3	12.0	14.9	33.5	19.7
2100	19.2	23.8	11.3	14.0	30.9	18.2
	18.5	23.0	10.9	13.5	29.9	17.6
	16.20	20.7	9.5	12.2	29.8	17.5
2130	18.5	22.3	10.9	13.1	30.3	17.8
	18.2	22.6	10.7	13.3	31.5	18.5
	18.9	23.1	11.1	13.6	33.0	19.4
2200	15.1	19.7	8.9	11.6	34.5	20.3
	18.5	22.6	10.9	13.3	35.0	20.6
	22.6	28.7	13.3	16.9	35.4	20.8
2230	26.5	31.1	15.6	18.3	35.2	20.7
	25.0	30.0	14.7	17.6	34.7	20.4
	25.0	29.6	14.7	17.4	33.8	19.9
2300	24.1	28.7	14.2	16.9	32.8	19.3
	23.1	26.9	13.6	15.8	31.1	18.3
	23.5	28.0	13.8	16.5	29.4	17.3
2330	24.1	28.7	14.2	16.9	26.9	15.8
	22.3	26.9	13.1	15.8	24.3	14.3
	18.9	24.1	11.1	14.2	21.6	12.7
2400	17.3	22.0	10.2	12.9	19.4	11.4
					17.3	10.2
					15.8	9.3
0030					14.5	8.5

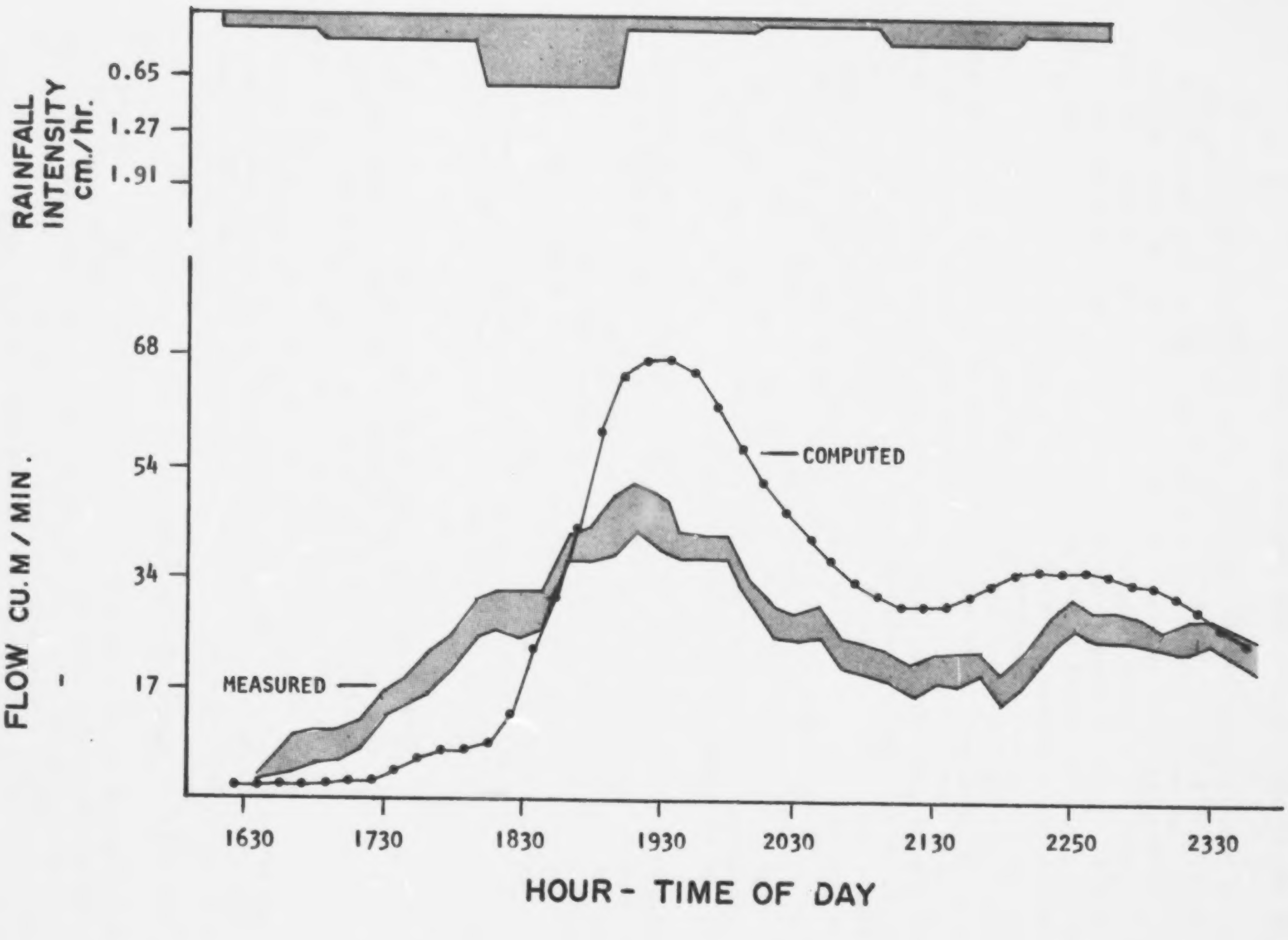


Figure 71. Run Number 21 arriving flow Site I.

goodness of fit of the two curves is generally acceptable since there is only a slight lag of the computed peak behind the measured.

The series samples collected during this run were taken manually since the treatment unit at Site I was operating automatically on the diverted dry weather flow when this rainfall started. Because of this operational problem, no discrete samples were taken during the first two hours of the overflow. After this delay, samples of the arriving flow and plant effluent were taken at 10 to 15 minute intervals. Table 80 lists the computed and measured quality values of the arriving flow. No coliform analyses were performed on these samples. Figure 72 presents a graphical comparison of the computed and actual concentrations and shows that the computed BOD values were much higher than the measured throughout the run. A review of the pertinent input data was conducted to determine where calibrations could be made. The only possibility was with the data describing the contents of each catchbasin in the area. The BOD of the stored contents of each catchbasin was initially estimated as 400 mg/l. After this run with the SWMM, a few of the catchbasin in the area were sampled and composited and the results indicated a BOD value of 150 mg/l. It was then decided to change the BOD value that is used as input to Runoff block to 150 mg/l and to rerun the SWMM. The results of this change are listed in Table 81 and plotted in Figure 73. This change brought the BOD values closer to the measured and the resulting goodness of fit is now excellent for both the BOD and suspended solids pollutographs.

The effluent from the treatment unit at Site I was also discretely sampled for comparison with the output of the Storage block. Table 82 lists the computed and actual effluent concentrations while Figure 74 presents the graphical comparison. Since the arriving quality comparisons were so close during this run, the resulting Storage block output provides a good check of the SWMM's ability to simulate the removals from the treatment units. As Figure 75 indicates, the goodness of fit of the two curves is excellent throughout the simulation. The Storage block is, therefore, accurate in its predictive capabilities.

The computed arriving flow at Site II was not affected by the sewer maintenance at Site I since the Transport network of the SWMM was modified to fit the existing conditions. Table 83 lists the actual and computed flows arriving at Site II and Figure 75 presents the graphical comparison of the flows. The ratio of the total measured flow arriving to the total computed arriving flow is 1.06. The computed peak is much lower in magnitude than the measured peak. This difference could be due to the mechanical problems in determining when and if the sluice gate in element 146, Figure 62, closed to channel more flow into this site. The goodness of fit of the two curves is acceptable throughout the run.

The arriving quality at this Site is determined by eight series samples taken during one hour of the overflow. Table 84 lists the measured and computed values for this run and Figure 76 presents the graphical comparison. The measured values show a definite peak in the BOD and suspended solids concentration which is preceded by the computed peaks of almost equal magnitude. The goodness of fit of the two curves is generally acceptable.

TABLE 80. QUALITY OF ARRIVING FLOW, SITE 1

Run Number 21
Catchbasin BOD = 400 mg/l

	Actual		Computed	
Time hours	BOD mg/l	SS mg/l	BOD mg/l	SS mg/l
1800			232	297
			235	313
			230	292
1830			222	355
			234	453
			232	418
1900			230	384
			232	395
			240	407
1930	74	300	240	408
	78	160	236	399
	99	280	229	370
2000			220	329
			210	286
			202	247
2030			194	212
	84	220	187	180
	107	340	180	153
2100			174	130
	112	310	168	112
	76	275	160	100
2130	72	190	155	96
	77	173	152	94
	54	115	149	94
2200			147	95
	103	288	145	97
			143	96
2230	59	230	141	95
			138	94
			135	93
2300			132	91
	56	86	130	89
			129	86
2330	53	90	128	83
			125	78
			123	73
2400			120	67
			118	61
			116	56
			115	51
			115	47

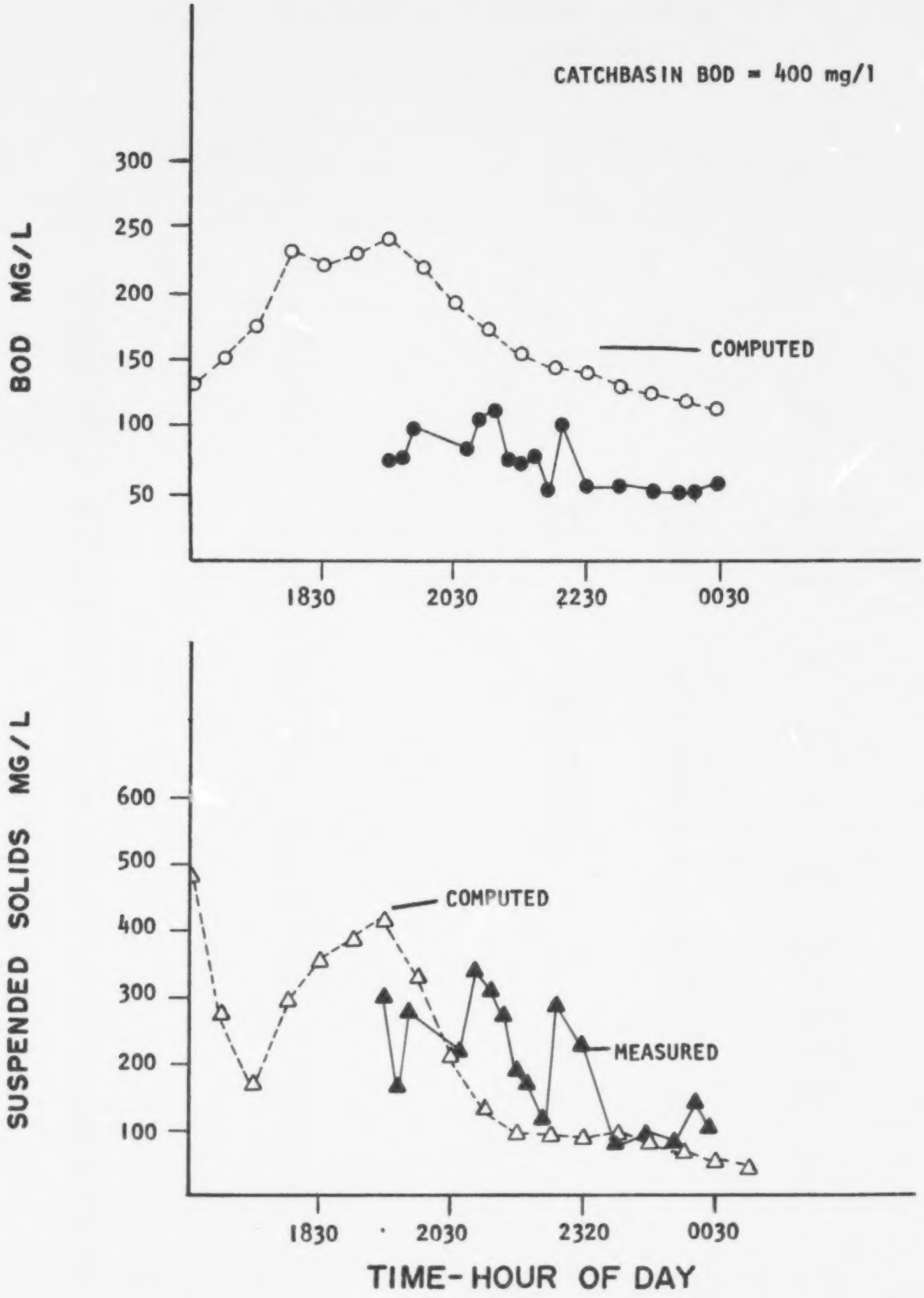


Figure 72. Run Number 21 arriving quality Site I.

TABLE 81. ARRIVING QUALITY, SITE I

Run Number 21
Catchbasin BOD = 150 mg/l

Time	ACTUAL		COMPUTED	
hours	BOD mg/l	SS mg/l	BOD mg/l	SS mg/l
1800				
			118	310
			117	306
			112	302
1830			114	422
			113	444
			108	397
1900			106	389
			107	411
			108	415
1930	74	300	107	415
	78	160	104	395
	99	280	100	358
2000			95	315
			91	272
			87	234
2030			83	200
	84	220	79	170
	107	340	76	144
2100			74	123
	112	310	70	106
	76	275	68	97
2130	72	190	67	95
	77	173	66	94
	54	115	65	94
2200			64	96
	103	288	63	96
			62	95
2230	59	230	61	94
			60	93
			58	91
2300			57	89
	156	86	57	86
			56	83
2330	53	90	55	79
	50	80	54	74
2400			53	69
	52	148	52	63
			51	57
0030	53	105	51	52

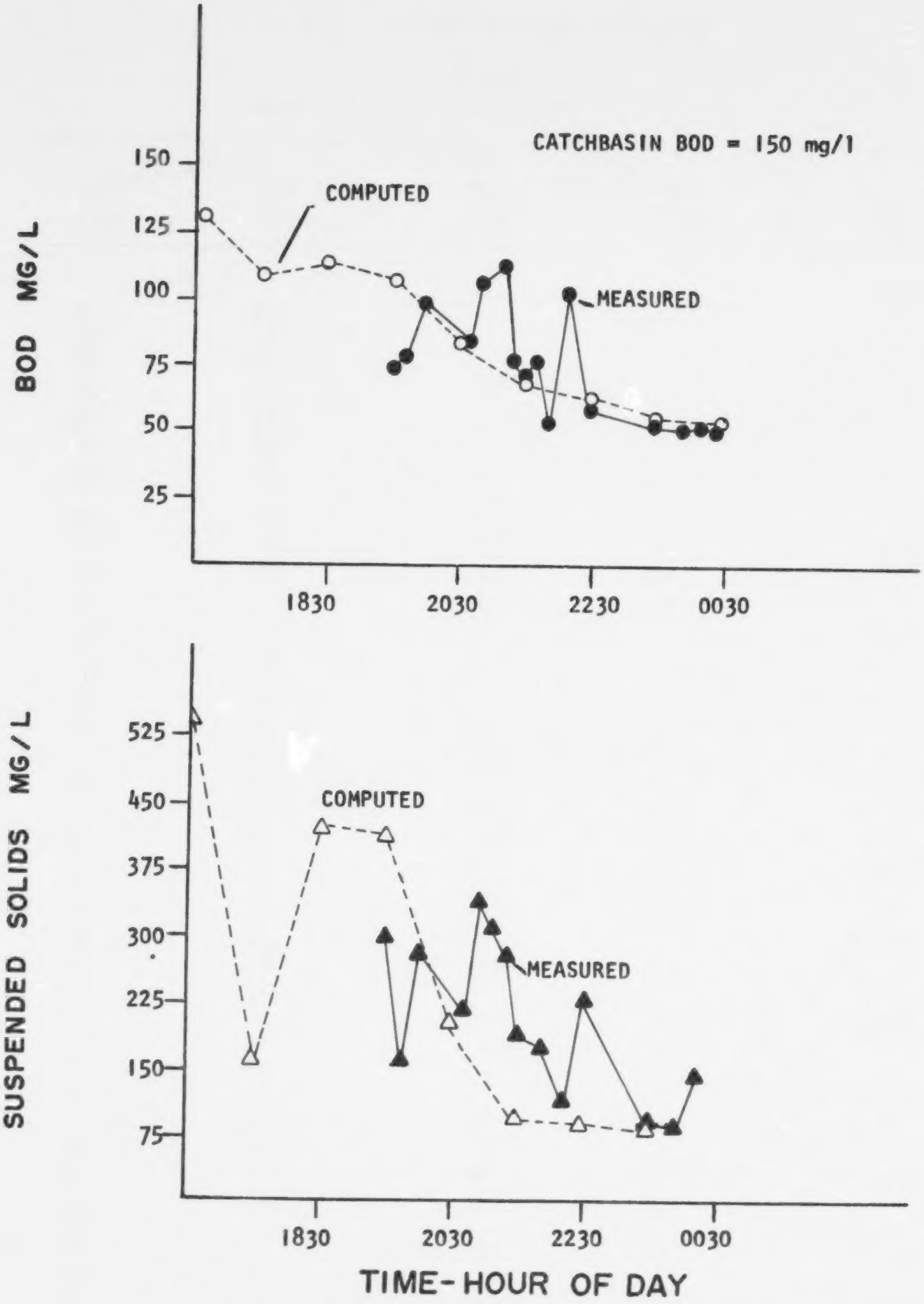


Figure 73. Run Number 21 arriving quality Site I.

TABLE 82. EFFLUENT QUALITY, SITE I

Run Number 21

Time hours	Actual Effluent BOD mg/l	Actual Effluent SS mg/l	Storage BOD mg/l	Storage SS mg/l
1800			49	50
			49	57
			48	56
1830			46	55
			47	91
			47	123
1900			47	142
			45	139
			46	146
1930	54	154	46	147
	49	158	46	147
	46	112	45	141
2000	37	84	43	130
	29	72	41	116
	31	86	39	102
2030	37	75	37	89
			35	77
	34	72	33	63
2100			32	50
	31	64	30	40
			29	34
2130	37	70	28	31
			28	30
	38	90	27	31
2200			27	32
	32	71	26	34
			26	34
2230	32	60	26	34
			25	34
	25	49	25	33
2300			24	32
	21	41	24	30
			24	28
2330	36	63	23	26
			23	23
	28	53	22	20
2400	25	52	22	17
	31	55		

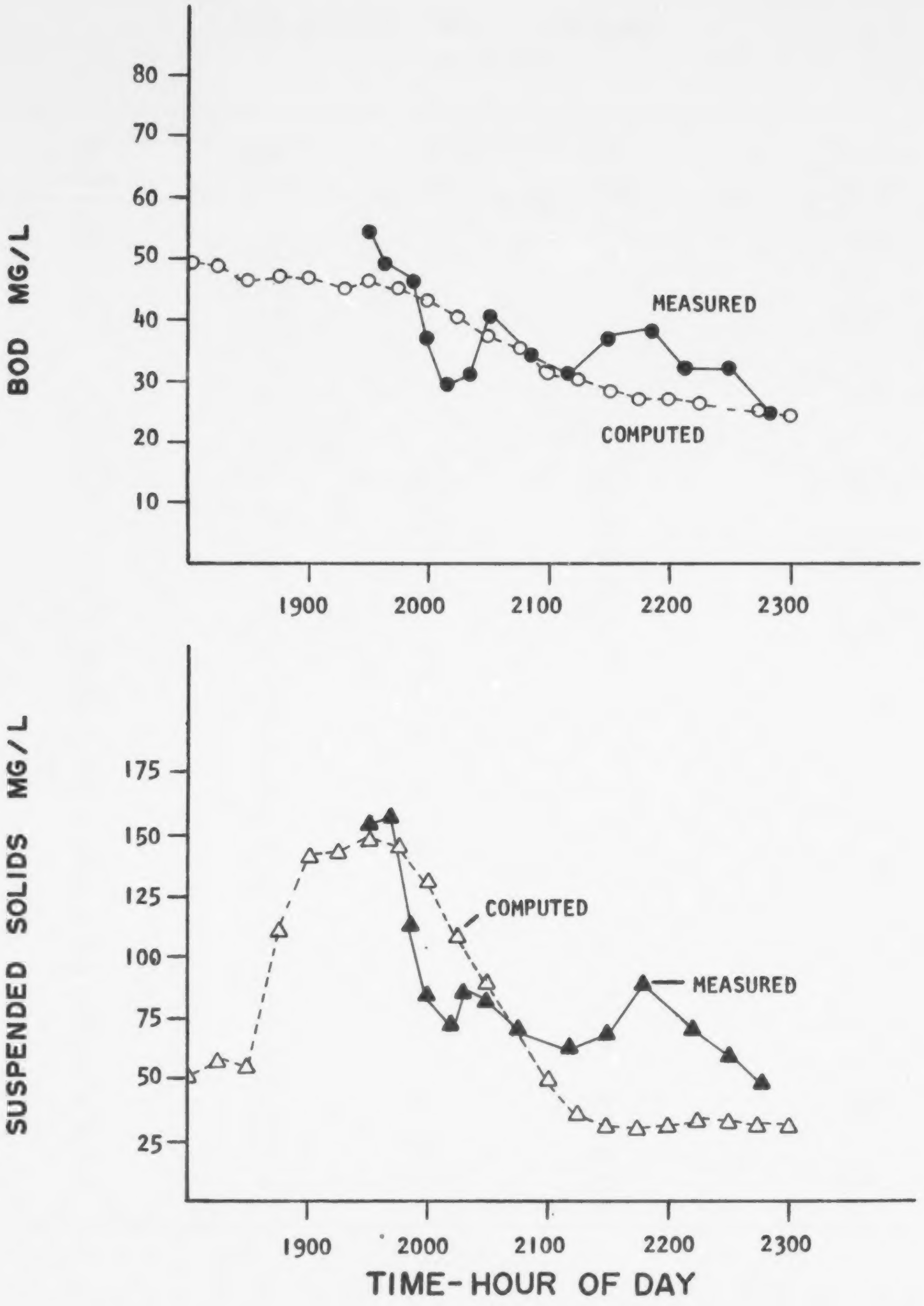


Figure 74. Run Number 21 effluent quality Site 1.

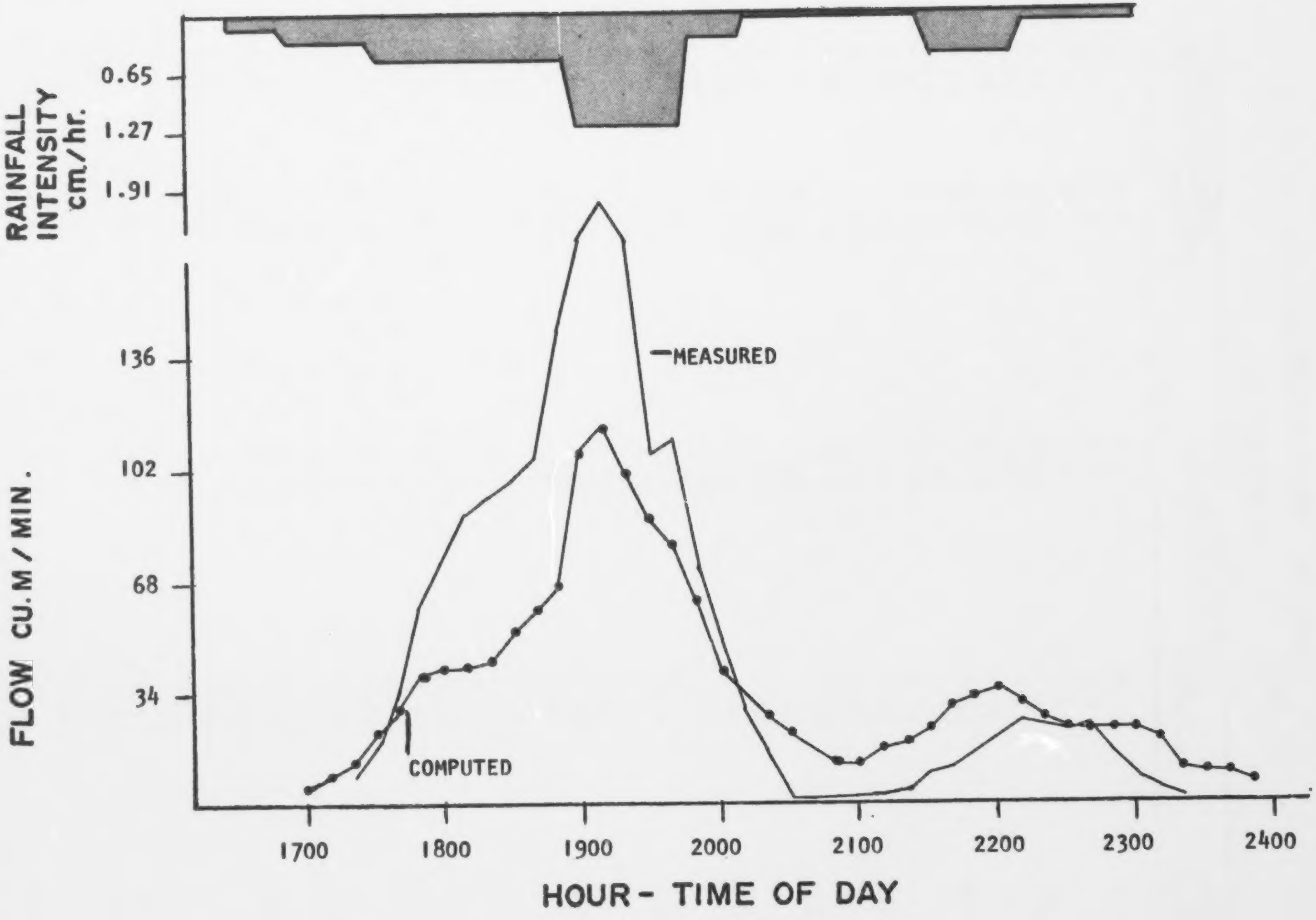


Figure 75. Run Number 21 arriving flow Site 11.

TABLE 83. ARRIVING FLOW, SITE II

Run Number 21

Time hours	Arriving flow, cu m/min	Arriving flow, cfs	Computed flow, cu m/min	Computed flow, cfs
1700	6.0	3.5	0.0	0.0
	8.3	4.9	0.0	0.0
	13.4	7.9	7.8	4.6
1730	21.1	12.4	19.7	11.6
	29.9	17.6	34.0	20.0
	38.4	22.6	60.7	35.7
1800	40.8	24.0	74.8	44.0
	41.3	24.3	88.4	52.0
	42.2	24.8	94.7	55.7
1830	51.7	30.4	98.6	58.0
	59.5	35.0	105.4	62.0
	65.6	38.6	143.1	84.2
1900	105.9	62.3	170.0	100.0
	114.1	67.1	181.9	107.1
	101.5	59.7	169.3	99.6
1930	86.7	51.0	105.6	62.1
	78.4	46.1	109.7	64.5
	61.9	36.4	71.9	42.3
2000	38.8	22.8	50.2	29.5
	31.3	18.4	27.9	16.4
	26.4	15.5	15.0	8.8
2030	20.6	12.1	1.9	1.1
	16.5	9.7	1.7	1.0
	12.8	7.5	0.0	0.0
2100	12.6	7.4	0.0	0.0
	12.9	7.6	2.2	1.3
	16.0	9.4	3.4	2.0
2130	18.2	10.7	7.7	4.5
	21.8	12.8	10.7	6.3
	28.6	16.8	15.5	9.1
2200	31.3	18.4	20.1	11.8
	33.0	18.4	24.3	14.3
	28.7	16.9	23.8	14.0
2230	24.8	14.6	23.1	13.6
	23.5	13.8	21.4	12.6
	23.0	13.5	14.6	8.6
2300	23.0	13.5	7.3	4.3
	21.9	12.9	3.6	2.1
	17.6	10.4	3.4	2.0
2330	9.9	5.8	1.0	0.6
	8.5	5.0	0.0	0.0
	6.3	3.7	0.0	0.0
2400	4.8	2.8	0.0	0.0

TABLE 84. ARRIVING QUALITY, SITE II

Run Number 21

	Actual		Computed	
Time hours	BOD mg/l	SS mg/l	BOD mg/l	SS mg/l
1800			122	890
			119	486
			114	285
1830			110	280
			110	369
			105	470
1900			96	510
			94	506
			91	843
1930	79	705	82	488
	33	188	73	541
	35	105	69	534
2000			66	521
			59	560
			56	555
2030			55	496
	30	110	53	435
	21	120	51	392
2100			49	314
	18	140	48	221
	21	133	48	173
2130	20	72	50	145
			48	120
			46	88
2200			45	76
			44	67
			44	69
2230			44	78
			42	82
			41	92
2300			40	106

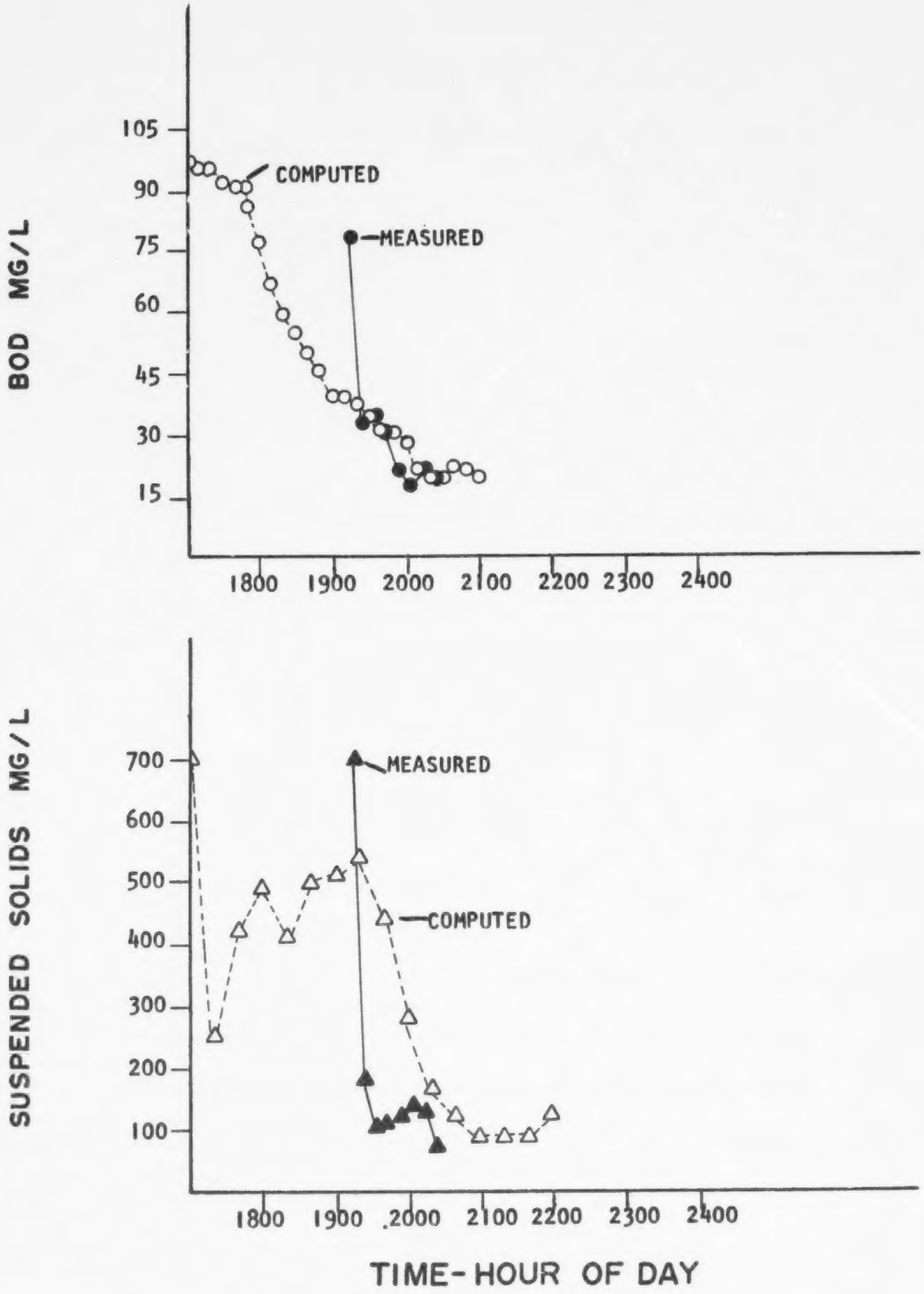


Figure 76. Run Number 21 arriving quality Site 11.

Run Number 25

The next rainfall event that was used to evaluate the SWMM output was run 25 which occurred on April 18, 1974. The total rainfall of this storm was 0.66 centimeters (0.26 inches) over 185 minutes. Table E4 of Appendix VI-E presents the rainfall intensities at five minute intervals for each site during this run. There were four days prior to this run in which the cumulative rainfall was less than 2.54 centimeters (1.0 inch). Table 85 presents the actual arriving flow at the wetwell of Site I and the computed flow for the same location. Figure 77 shows the graphical comparison of these two flows. The actual flow starts almost one hour earlier than the computed, and peaks 30 minutes later than the computed. The ratio of the total actual flow arriving to the computed total flow is 1.34. The closeness of fit of the two curves is generally fair. The reason the computed flow peaks early could be due to the slopes of the conveyance system used in the SWMM, or to the inaccuracies of the timing mechanisms of the rain gauges. The input variable which describes the percent of the impervious area with zero detention was increased to determine if this would bring the peaks closer together. Because the remaining runs varied in the time of arrival of the peak, some were early and other behind the actual, and because run No. 21 was accurate, it was decided to leave the default value of 25 percent.

The quality of the arriving flow was determined by nine samples taken over the duration of the overflow and analyzed for BOD, suspended solids and total coliforms. The number of total coliforms is expressed as number per 100 ml as determined by the membrane filter technique while the computed value is expressed as MPN per 100 ml. Although the two methods are different, it was found, by running both methods on the same samples, the two results were within the same acceptable range. The remaining coliform analyses were run using the membrane filter technique. Table 86 presents the actual measured quality and the computed quality of the arriving flow at Site I. The actual samples were taken as the overflow began (1500 hours) and at various times throughout the overflow. Figure 78 presents the graphical comparison of the BOD and suspended solids data. The measured BOD and suspended solids are relatively constant at 40 and 150 mg/l respectively. The absence of a "first flush" could be due to the occurrence of over 2.54 centimeters (1 inch) of rain four days earlier. The computed BOD is reasonably close to the measured although a difference of 20 mg/l is present at the peak of the computed pollutograph. The computed suspended solids pollutograph peaks well above the measured, but without this initial peak the computed values are close to the actual measured concentrations. Figure 79 presents the arriving coliform data which is very close in comparison. In this case, the computed arriving flow for this run is 30 minutes earlier than the actual, therefore if it were brought closer to the actual, it might change the quality predictions which show acceptable goodness of fit.

The effluent from the Site I treatment unit was also discretely sampled during the overflow and the measured results along with the computed output of the Storage block are shown in Table 87. No coliform analyses

TABLE 85. ARRIVING FLOW, SITE I

Run Number 25

Time	Arriving flow,				Computed flow,	
	cu m/min		cfs		cu m/min.	cfs
hours	min.	max.	min.	max.		
1500	0.9	1.7	0.5	1.0		
	1.5	2.2	0.9	1.3		
	0.9	1.2	0.5	0.7		
1530	1.4	1.7	0.8	1.0		
	1.7	2.4	1.0	1.4		
	2.0	2.9	1.2	1.7		
1600	3.4	3.7	2.0	2.2	1.2	0.7
	2.9	6.0	1.7	3.5	1.0	0.6
	2.9	6.0	1.7	3.5	7.1	4.2
1630	3.4	6.8	2.0	4.0	13.1	7.7
	5.3	8.3	3.1	4.9	17.7	10.4
	12.8	15.1	7.5	8.9	20.2	11.9
1700	21.9	22.6	12.9	13.3	21.4	12.6
	26.5	29.6	15.6	17.4	20.7	12.2
	27.9	31.8	16.4	18.7	18.0	10.6
1730	27.9	31.8	16.4	18.7	15.1	8.9
	24.1	26.5	14.2	15.6	12.2	7.2
	21.9	23.5	12.9	13.8	9.7	5.7
1800	12.8	15.1	7.5	8.9	7.7	4.5
	8.3	11.6	4.9	6.8	5.8	3.4
	9.7	14.3	5.7	8.4	4.3	2.5
1830	6.8	9.0	4.0	5.3	2.9	1.7
					1.7	1.0
					0.7	0.4

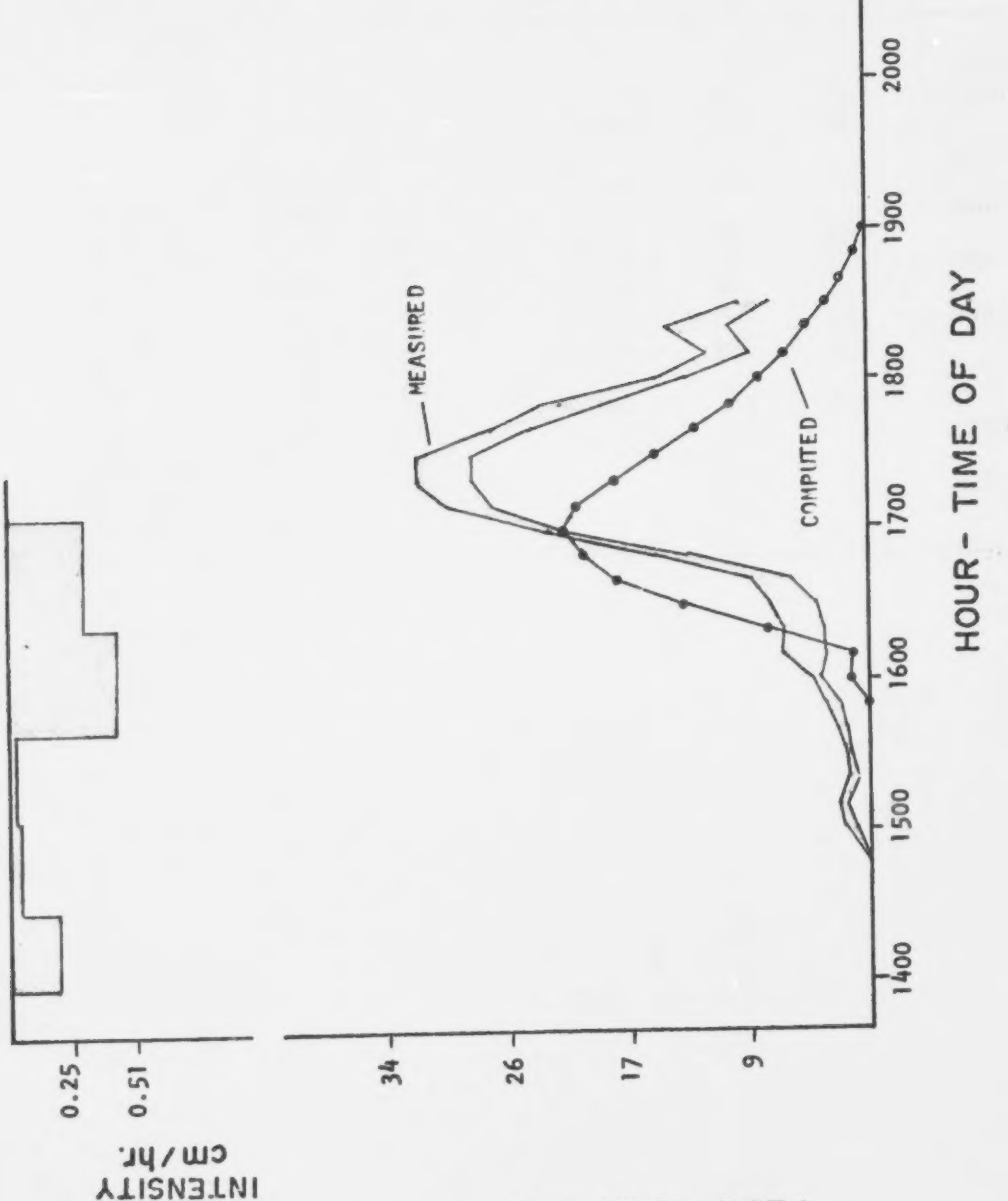


Figure 77. Run Number 25 arriving flow Site 1.

TABLE 86. ARRIVING QUALITY, SITE I

Run Number 25

Time hours	Actual BOD mg/l	Actual SS mg/l	Actual Coliforms No./100 ml	Computed BOD mg/l	Computed SS mg/l	Computed Coliforms MPN/100 ml
1455						
1500	56	93	1.1×10^7			
	52	134	1.4×10^6			
1530						
	30	129	1.0×10^6			
1600				50.9	444.0	1.4×10^5
	56	84	2.6×10^5	77.0	190.5	5.1×10^6
1630	43	91	8.5×10^4	76.0	166.8	3.5×10^6
	31	98	2.7×10^6	65.8	182.3	2.3×10^6
1700	45	171	2.4×10^6	56.3	186.7	1.8×10^6
	53	230	1.5×10^6	46.6	175.6	1.9×10^6
1730				38.8	150.9	2.2×10^6
	33	212	1.4×10^6	33.1	123.4	2.7×10^6
1800				29.4	99.1	3.3×10^6
				27.0	79.9	4.1×10^6
1830				25.5	64.2	5.1×10^6
				25.0	58.2	6.1×10^6

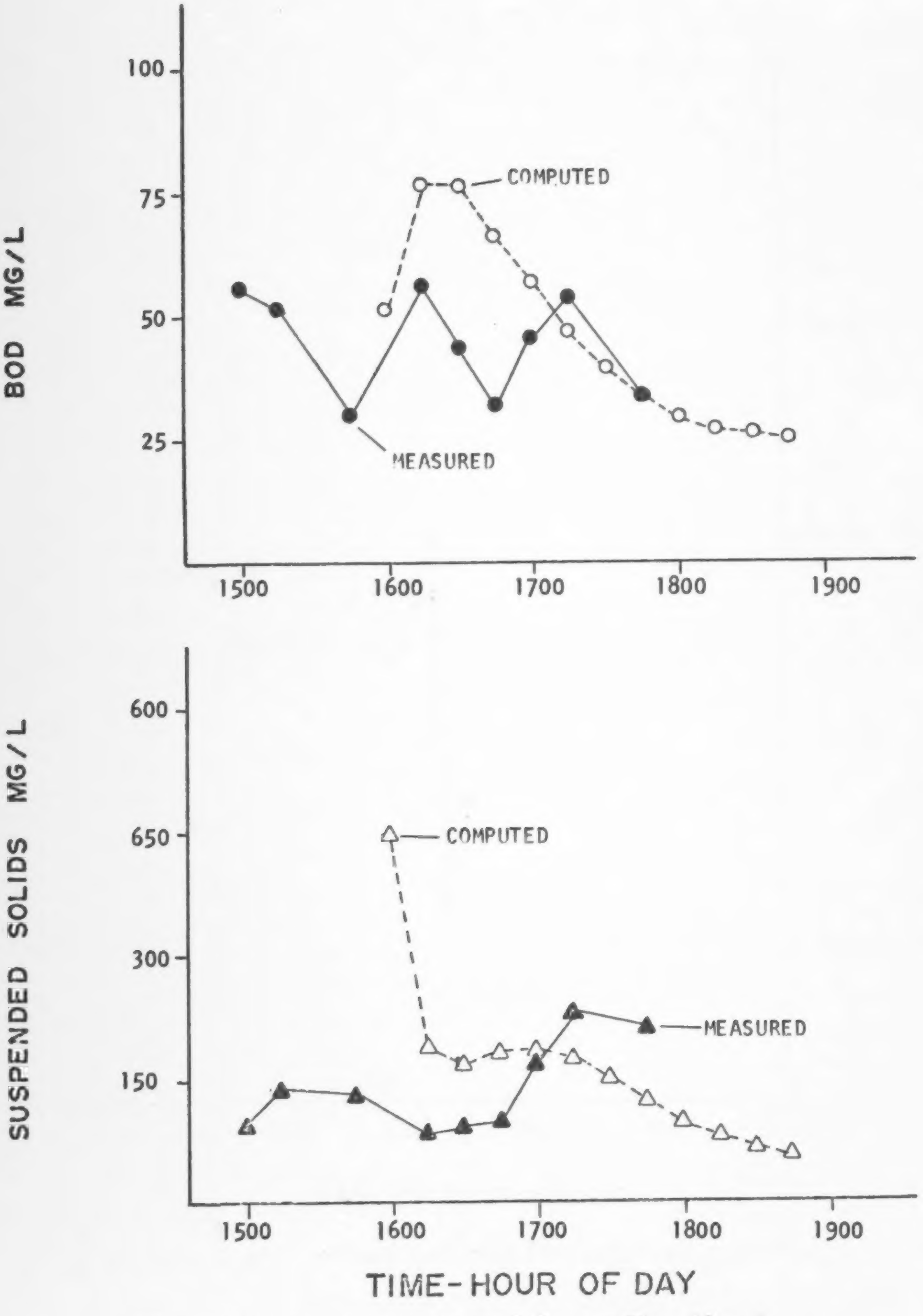


Figure 78. Run Number 25 arriving quality Site 1.

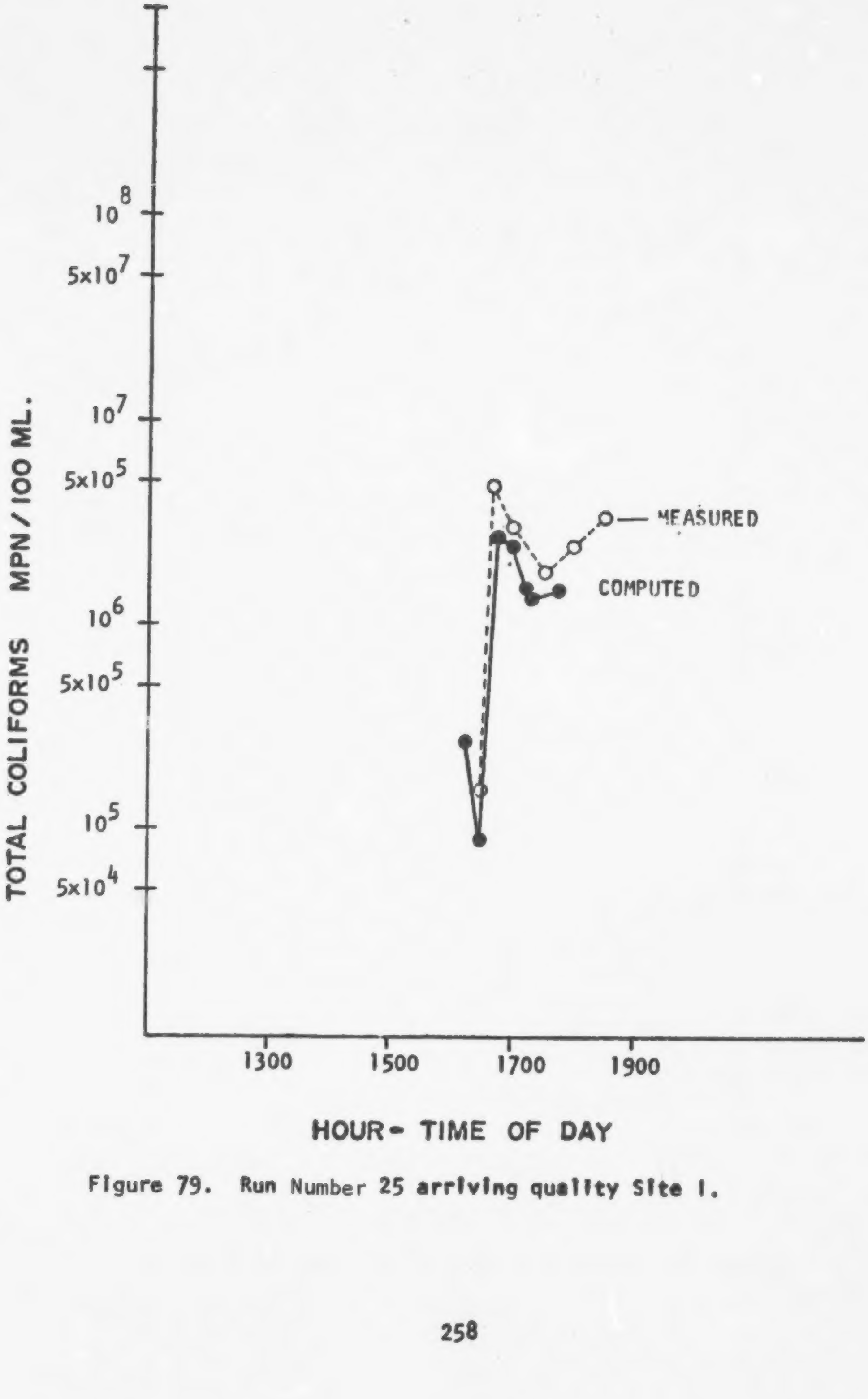


Figure 79. Run Number 25 arriving quality Site I.

TABLE 87. EFFLUENT QUALITY, SITE I

Run Number 25

	Actual		Computed	
Time hours	BOD mg/l	SS mg/l	BOD mg/l	SS mg/l
1500				
	126	267		
1530				
	33	59		
1600			26.9	176.3
	40	97	23.4	165.9
			21.0	155.4
1630	32	46	19.4	148.3
	37	58	18.5	143.6
			17.1	134.6
1700	35	62	27.9	57.8
	48	124	19.9	57.5
			17.4	70.8
1730	46	110	14.5	77.3
	32	81	11.7	76.5
			9.3	70.0
1800	30	55	7.3	60.6
			5.8	49.9
			4.6	38.5
1830			3.8	28.9
			3.4	20.0
			3.1	12.9
1900			3.0	8.0
			3.0	5.9
			3.0	4.5

were performed on these samples. Figure 80 presents the graphical comparison of these data. The first measured value of the effluent BOD and suspended solids is extremely high because of the scouring of solids from the bottom of the flotation tanks and the poor flotation at the very start of the effluent discharge. These high values may be discarded since they are higher in concentration than the influent throughout the duration of the overflow. To compare the remaining values, the influent quality must serve as a function of the computed effluent quality. Thus, large variations between the actual and computed influent values will make the comparison of the effluent quality difficult. The measured BOD of the effluent is generally higher than the computed, but since the arriving BOD is generally less than 50 mg/l, the treatment units could not remove a substantial part of the incoming BOD. The arriving computed BOD was greater than the measured value, while the computed effluent concentration was lower. The suspended solids data follow the same general pattern with the computed effluent values being lower in concentration than the measured values after being greater in concentration in the arriving flow. Again the low suspended solids concentration in the arriving flow could be the reason for the variations. In summation, the overall computed effluent quality shows good correlation throughout the overflow.

The actual and measured flows at Site II for run No. 25 are listed in Table 88. The computed values of the arriving flow are taken from two points in the transport system of the SWMM. This procedure is used whenever the sluice gate in element 146 of Figure 62 is closed. When this occurs, all flow in the interceptor is routed to Site II and the computed arriving flow is taken as element 87, not 112 as is the usual case. When the gate opens, the computed arriving flow is taken as element 112. The graphical comparison of the arriving flow is shown in Figure 81. Both the computed and measured hydrographs start and stop at the same time. Although there is no definite peak in the computed flow, the highest computed value occurs at about the same time as the measured peak. The area under each curve is very close with the ratio of measured to the computed areas being 0.96. The closeness of fit of the two hydrographs is poor even though the total flows arriving are almost equal.

The arriving flow at Site II was sampled ten times during the duration of the run. Table 89 lists the measured quality characteristics and the computed values. Figure 82 presents the graphical comparison of the measured and computed quality. The measured BOD and suspended solids are relatively constant except for the large peak at 1715 hours. At this point in time the corresponding hydrograph is also at a maximum. The computed values of the BOD and suspended solids decrease from the start of the overflow and do not show the peak which the measured values do. This peak could correspond to a "flush" phenomenon, but it has not been seen in other overflow events at this site. Figure 83 presents the arriving coliform data in graphical form and again the computed and measured values are very close throughout the overflow. Thus, the overall comparison of the arriving quality for this run is generally poor in the suspended solids and BOD constituents and very good with the coliform data. No discrete effluent

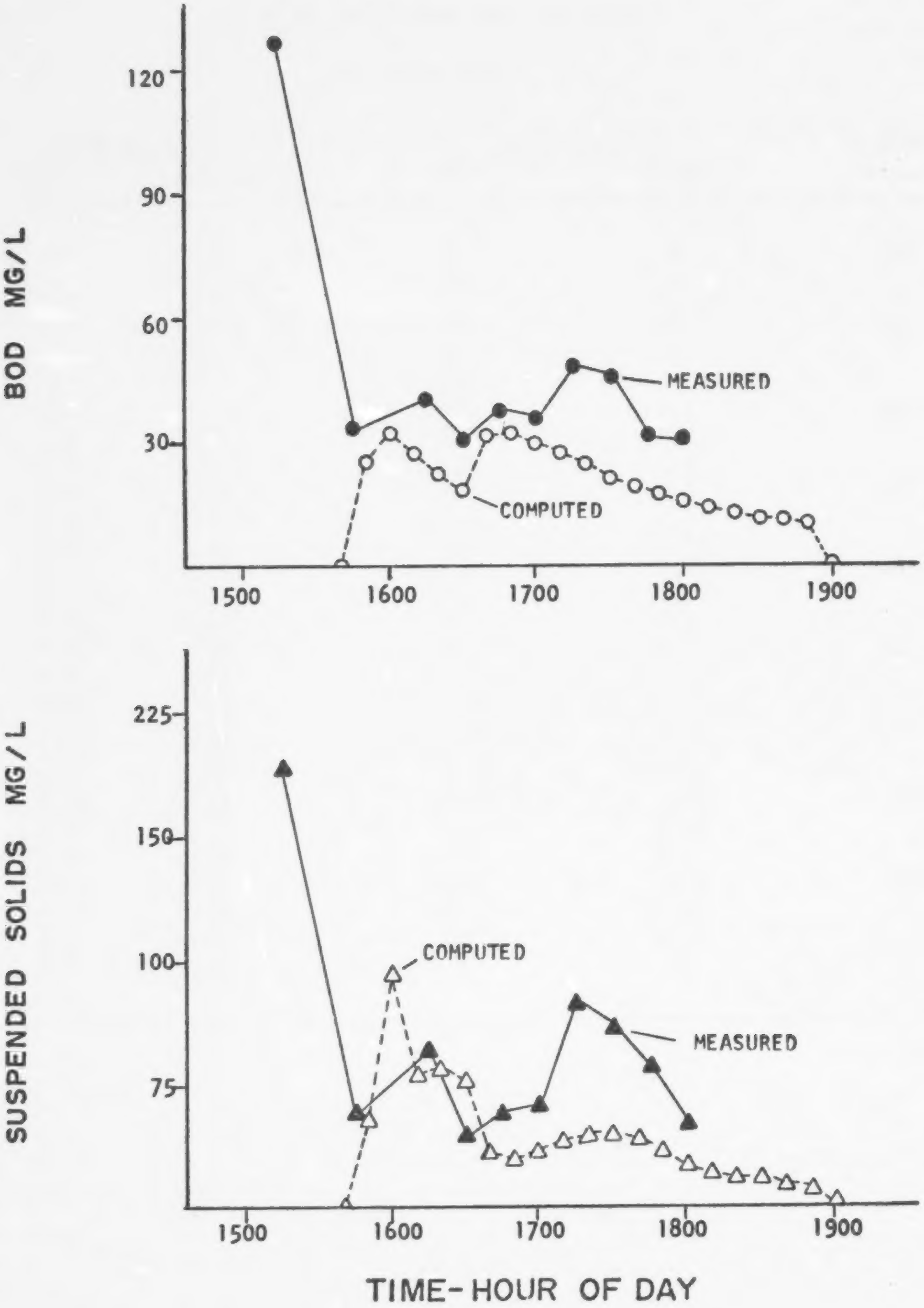


Figure 80. Run Number 25 effluent quality Site I.

TABLE 88. ARRIVING FLOW, SITE II

Run Number 25

Time hours	Arriving flow, cu m/min	Arriving flow, cfs	Computed flow, cu m/min	Computed flow, cfs
1400	0	0	0	0
	0	0	0	0
	0	0	0	0
1430	0	0	0	0
	0	0	0	0
	1.7	1.0	0	0
1500	3.4	2.0	0	0
	7.5	4.4	10.5	6.2
	8.7	5.1	10.7	6.3
1530	9.5	5.6	10.7	6.3
	10.5	6.2	10.7	6.3
	11.2	6.6	11.6	6.8
1600	10.9	6.4	19.0	11.2
	10.2	6.0	32.2	21.3
	9.5	5.6	42.0	24.7
1630	9.5	5.6	42.7	25.1
	13.1	7.7	44.2	26.0
	44.2	26.0	48.3	28.4
1700	109.0	64.1	51.0	30.0
	110.8	65.2	58.1	34.2
	80.2	47.2	56.4	33.2
1730	58.7	34.5	45.6	26.8
	18.9	11.1	27.2	16.0
	11.4	6.7	21.6	12.7
1800	11.4	6.7	19.4	11.4
	9.5	5.6	17.7	10.4
	7.5	4.4	15.0	8.8
	0	0	14.6	8.6
			0	0

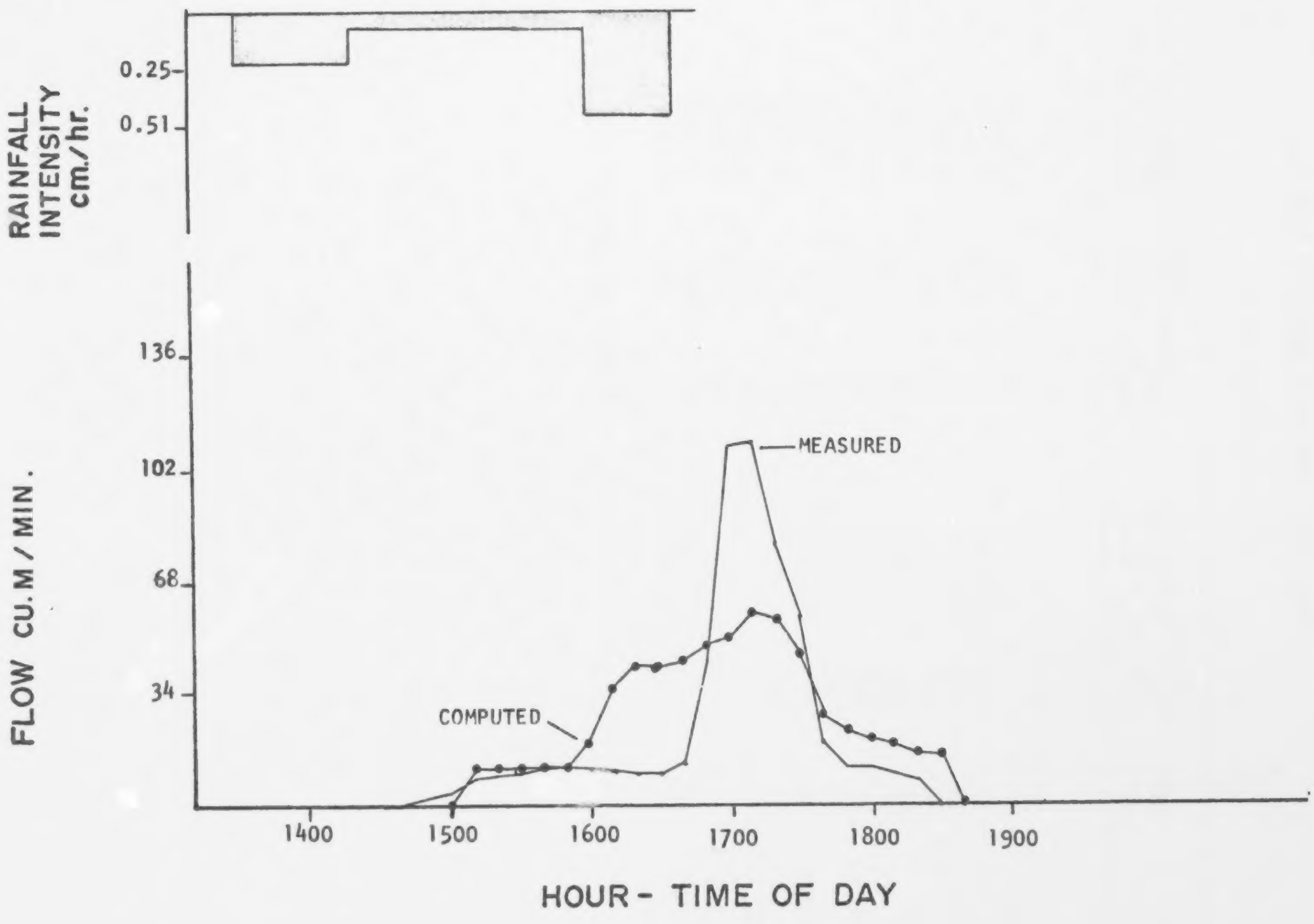


Figure 81. Run Number 25 flow Site 11.

TABLE 89. ARRIVING QUALITY, SITE 11

Run Number 25

Time hours	Actual BOD mg/l	Actual SS mg/l	Actual Coliforms No./100 ml	Computed BOD mg/l	Computed SS mg/l	Computed Coliforms MPN/100 ml
1500				77	104	6.9×10^6
	57	235	6.8×10^5	75	101	7.3×10^6
1530	54	233	9.0×10^6	72	161	4.6×10^6
	42	142	1.3×10^7	70	321	1.9×10^6
1600	62	156	1.2×10^6	58	352	1.1×10^6
	50	143	9.5×10^5	49	247	1.8×10^6
1630	42	134	2.5×10^5	45	239	1.4×10^6
	43	195	1.7×10^6	39	260	1.0×10^6
1700	58	293	8.0×10^5	39	244	1.3×10^6
	104	546	2.1×10^6	25	236	1.7×10^6
1730				20	225	3.3×10^6
				17	218	3.5×10^6
1800				21	207	5.0×10^6
	45	162	1.7×10^6	16	166	5.1×10^6
1830						

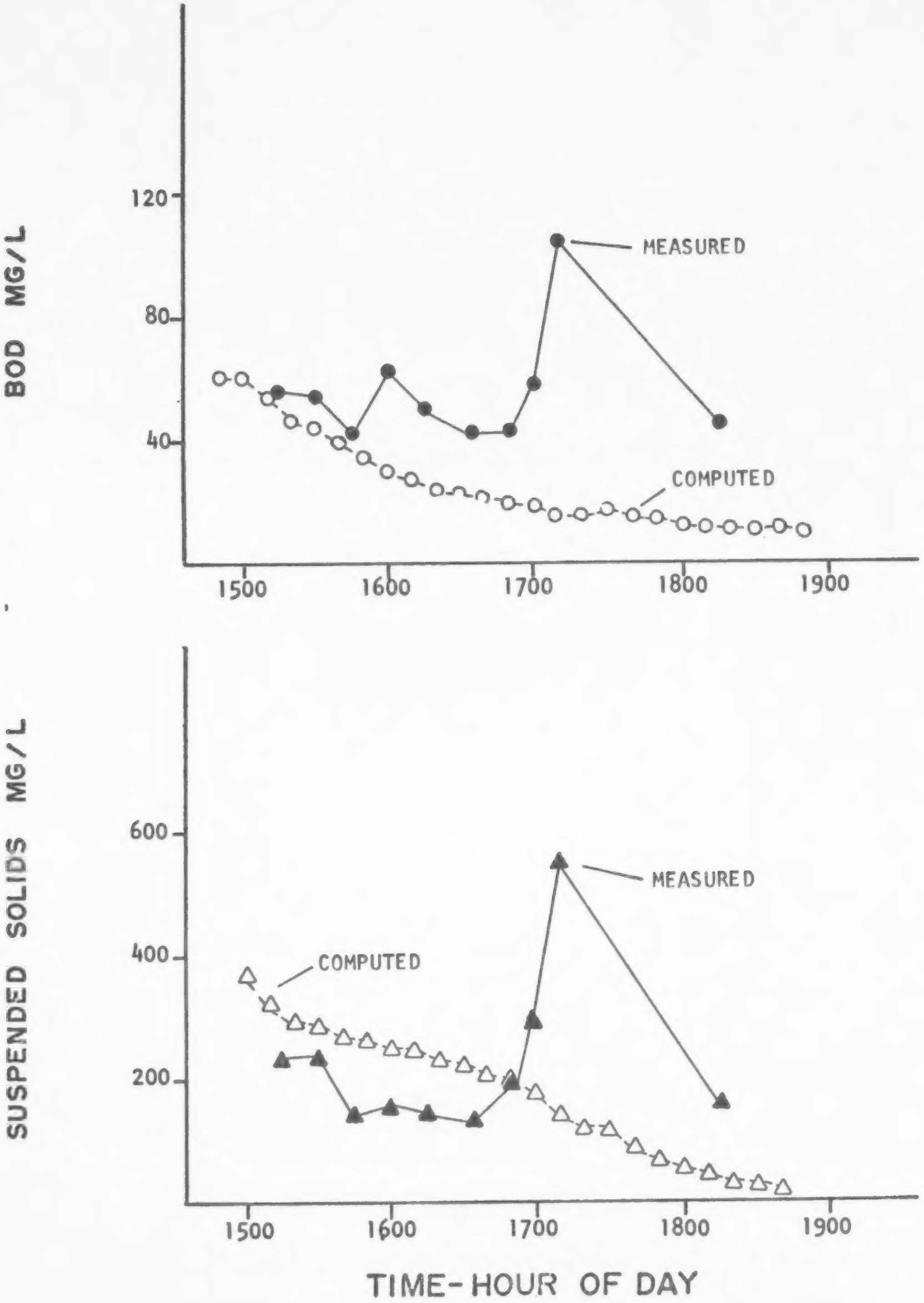


Figure 82. Run Number 25 arriving quality Site II.

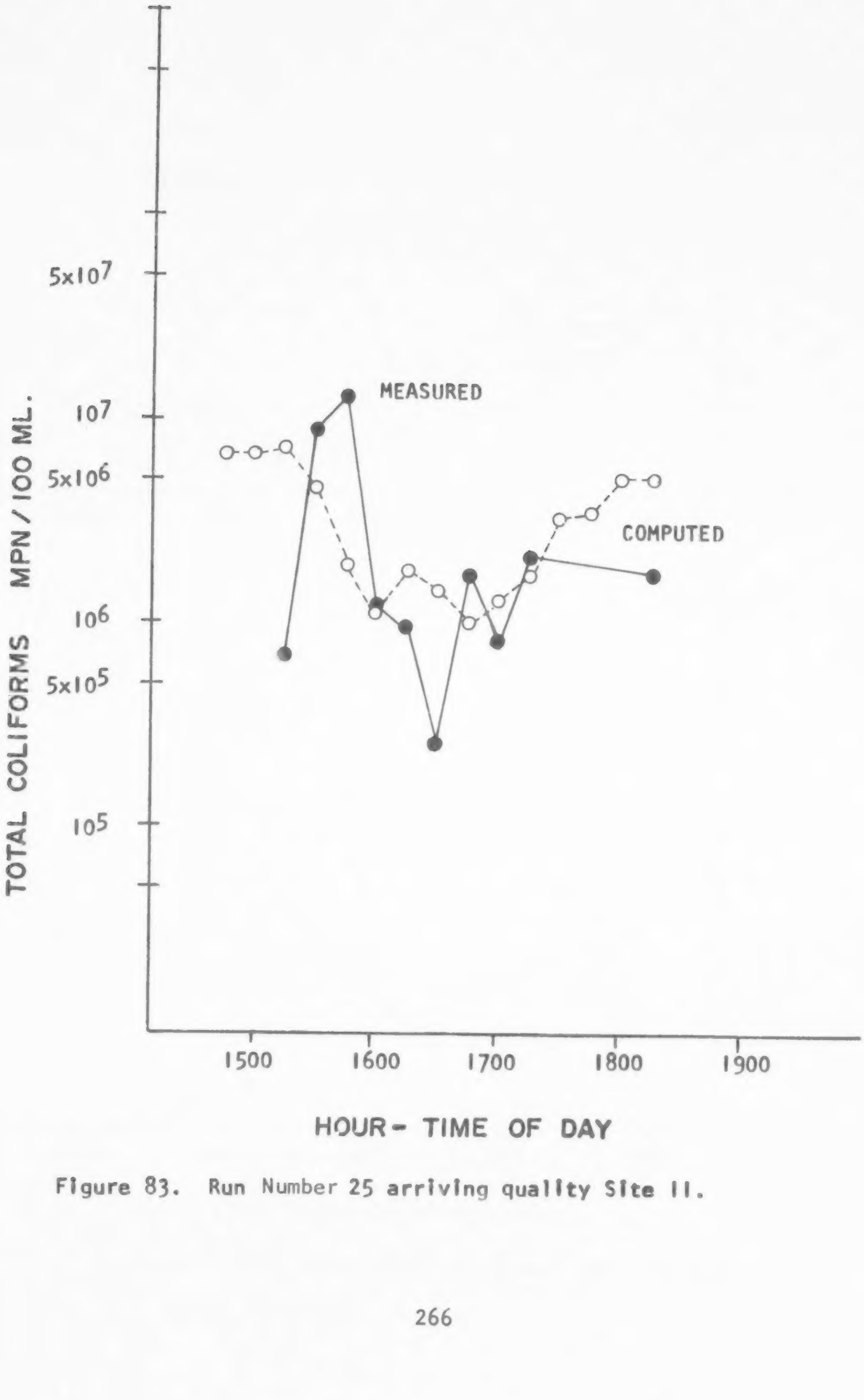


Figure 83. Run Number 25 arriving quality Site II.

samples from this site were taken due to mechanical problems with the chemical pumps of the treatment units.

Run Number 26

The third rainfall event in which discrete sample data were obtained is run Number 26 which occurred on April 28 and 29, 1974. The total rainfall was 3.46 centimeters (1.36 inches) contributing to Site I and 4.27 centimeters (1.68 inches) to Site II over a duration of 275 minutes. Table E5, Appendix VI-E presents the rainfall intensities at five minute intervals for each site. Prior to this run there were fifteen days in which the cumulative rainfall was less than 2.54 centimeters (1.0 inch) and seven days in which no rain fell. Table 90 presents the measured arriving flow and computed flows for Site I during this run. The measured arriving flow at this site is shown as ending at 0800 hours but the actual flow continued to arrive for more than 16 hours after this point. Figure 84 shows the graphical comparison of the computed and measured data. Note that the computed flow terminates shortly after 0400 hours. The large amount of rain could have caused excessive surcharging along the transport system but not enough to cause the more than 24 hours of overflow. The flow divider in subarea 18 was investigated to determine if its submergence was caused by the high levels of Lake Michigan during the Spring of 1974. No mechanical problems were found with this element and since the arriving flow at Site II did not show this same long duration, the possible surcharging of the Augusta Street subarea could be the cause. During this overflow event another agency was conducting flow measurements in selected sewers in the area and these measurements (40) for manhole 47 of the SWMM transport system are shown in Table 91. Figure 85 graphically presents the computed flows for this element during run Number 26. Note that the actual flows continue at high levels until well after the computed flows and that the computed flows surcharge from 0140 to 0320 hours at the full capacity of sewer number 72. Without this surcharging the computed flow would peak at approximately the same time as the measured flow. Thus, the goodness of fit of the flows for this run are very poor and the ratio of the actual to computed flow is less than 0.20.

The arriving quality for this run is not presented because of mechanical problems with the automatic samplers. The effluent quality is available and can be used to get a fair estimate of the overall quality predictive capacity of the SWMM for this run. Table 92 presents the computed and measured effluent quality for this run. The graphical comparison shown in Figure 86 shows the computed BOD and suspended solids data to be in the general range of the measured but the trends differ in the time the computed values terminate and peak. The coliform data are graphically presented in Figure 87. Again, the computed values are reasonably close to the measured. The goodness of fit of the quality data is fair.

Table 93 lists the measured and computed arriving flow for run No. 26 at Site II. The arriving flow ends shortly after 0600 hours. Figure 88 shows the graphical comparison of the measured and computed flows which are reasonably close throughout the duration of the run. The ratio of the total actual

TABLE 90. ARRIVING FLOW, SITE I

Run Number 26

Time hours	Arriving flow, cu m/min		Arriving flow, cfs		Computed flow,	
	min.	max.	min.	max.	cu m/min.	cfs
2350	27.2	30.3	16.0	17.8		
2400	29.6	32.3	17.4	19.0		
	29.6	32.3	17.4	19.0		
	28.1	33.3	16.5	19.6		
0030	30.9	34.9	18.2	20.5	0.0	0.0
	31.8	34.9	18.7	20.5	18.7	11.0
	34.0	38.6	20.0	22.7	37.9	22.3
0100	37.7	45.6	22.2	26.8	60.5	35.6
	43.7	51.7	25.7	30.4	75.8	44.6
	47.8	55.8	28.1	32.8	85.2	50.1
0130	50.3	58.3	29.6	34.3	87.2	51.3
	50.8	60.5	29.9	35.6	85.2	50.1
	50.3	58.3	29.6	34.3	79.6	46.8
0200	48.8	56.8	28.7	33.4	75.8	44.6
	47.8	55.8	28.1	32.8	71.9	42.3
	46.8	55.3	27.5	32.5	70.4	41.4
0230	46.4	54.4	27.3	32.0	66.3	39.0
	45.2	53.2	26.6	31.3	64.4	37.9
	44.7	52.3	26.3	31.0	60.5	35.6
0300	43.7	51.0	25.7	30.0	56.8	33.4
	42.7	50.1	25.1	29.8	53.0	31.2
	41.7	49.6	24.5	29.2	56.1	33.0
0330	42.5	50.5	25.0	29.7	53.0	31.2
	41.7	49.6	24.5	29.2	49.3	29.0
	40.8	48.8	24.0	28.7	41.7	24.5
0400	38.8	46.8	22.8	27.5	34.0	20.0
	36.7	44.7	21.6	26.3	22.1	13.0
	34.0	42.0	20.0	24.7	0.0	0.0
0430	34.0	42.0	20.0	24.7		
	34.0	42.0	20.0	24.7		
	34.0	42.0	20.0	24.7		
0500	34.0	42.0	20.0	24.7		
	26.6	30.3	15.6	17.8		
	22.7	28.1	13.3	16.5		
0530	21.9	25.7	12.9	15.1		
	26.6	26.5	13.3	15.6		
	23.8	27.5	14.0	16.2		
0600	26.0	30.3	15.3	17.8		
	26.5	30.3	15.6	17.8	0	0
	26.9	30.9	15.8	18.2	0	0
0630	28.1	31.8	16.5	18.7	0	0
	28.1	32.1	16.5	18.9	0	0
	28.1	32.1	16.5	18.9	0	0
0700	28.1	32.1	16.5	18.9	0	0
	28.1	32.1	16.5	18.9	0	0
	28.1	32.1	16.5	18.9	0	0
0730	28.1	32.1	16.5	18.9	0	0
	28.1	32.1	16.5	18.9	0	0
	28.1	32.1	16.5	18.9	0	0
0800	26.5	30.3	15.6	17.8	0	0

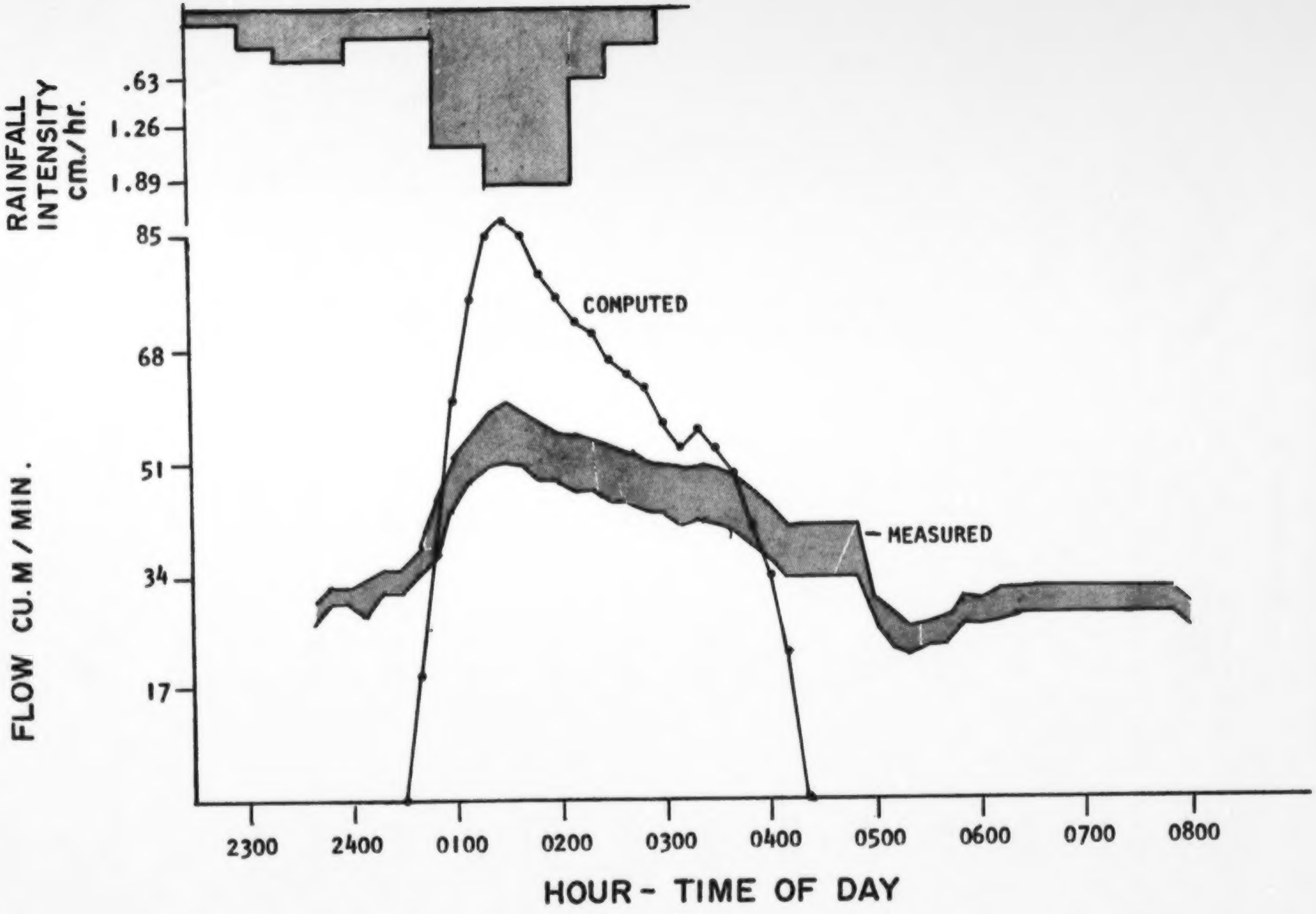


Figure 84. Run Number 26 arriving flow Site 1.

TABLE 91. UPSTREAM FLOW MEASUREMENTS

Run Number 26

Time hours	Measured Flow		Computed Flow		
	cu m/min	cfs	cu m/min	cfs	
2230	4.1	2.4			
2300	11.6	6.8			
2330	11.6	6.8			
			6.1	3.6	
			8.8	5.2	
2400	19.0	11.2	12.6	7.4	
			22.3	13.1	
			29.8	17.5	
0030	48.8	28.7	34.2	20.1	
			36.7	21.6	
			38.3	22.5	
0100	91.6	53.9	40.6	23.9	
			53.7	31.6	
			62.1	36.5	
0130	113.2	66.6	71.4	42.0	
			73.1	43.0	Surcharge
			73.1	43.0	
0200	52.7	31.0	73.1	43.0	
			73.1	43.0	
			73.1	43.0	
0230	46.8	27.5	73.1	43.0	
			73.1	43.0	
			73.1	43.0	
0300	42.7	25.1	73.1	43.0	
			73.1	43.0	
			73.1	43.0	
0330	38.9	22.9	78.2	46.0	
			65.0	38.2	
0400	21.1	35.9	53.4	31.4	
			45.6	26.8	
			19.9	11.7	
0430	19.3	32.8	17.2	10.1	
			16.7	9.8	

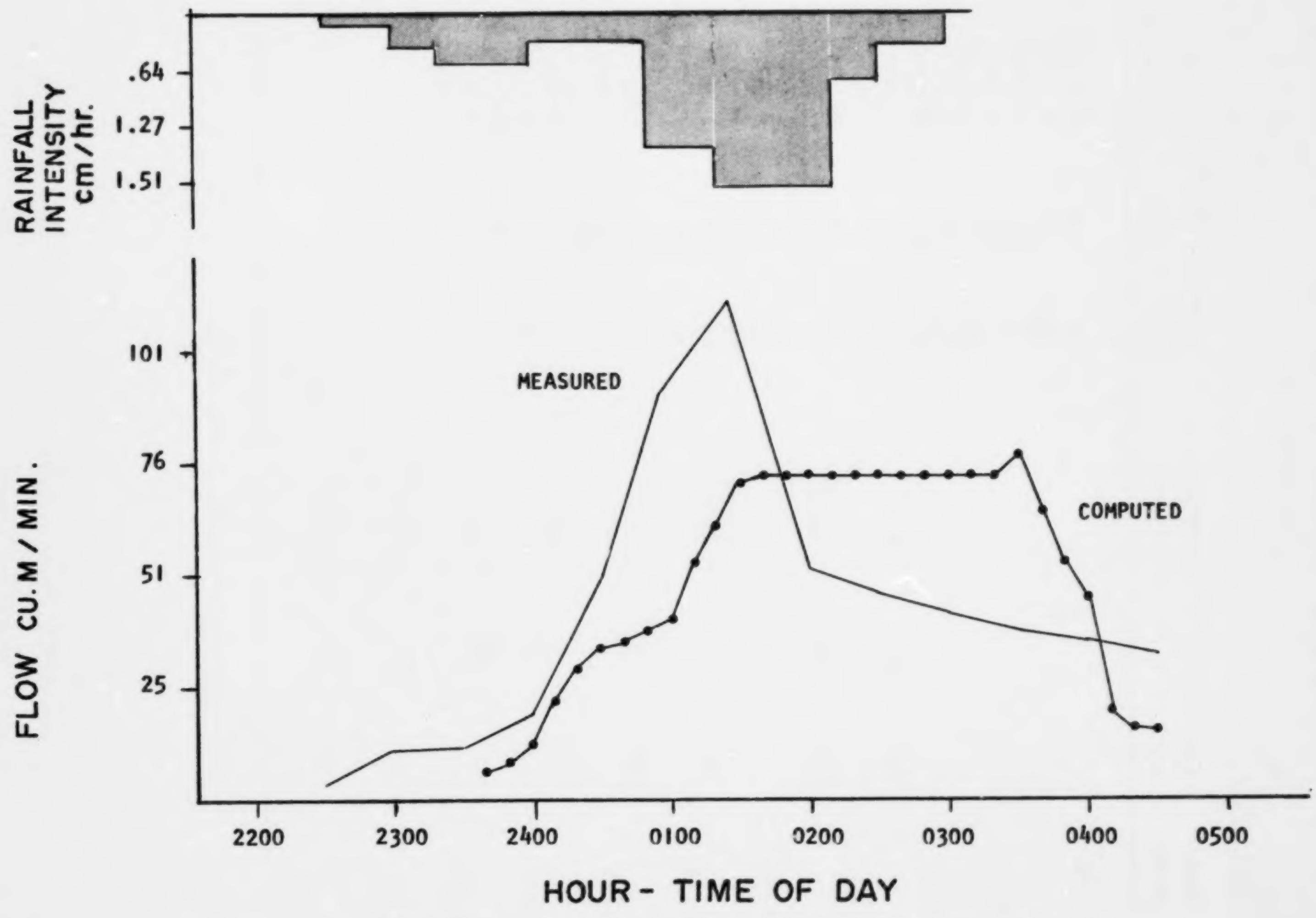


Figure 85. Run Number 26 upstream flow at element 47.

TABLE 92. EFFLUENT QUALITY, SITE I

Run Number 26

	Actual			Computed		
Time hours	BOD mg/l	SS mg/l	Coliforms No./100 ml	BOD mg/l	SS mg/l	Coliforms MPN/100 ml
2300				0	0	1.5×10^1
				39	27	4.1×10^1
				36	75	1.4×10^2
2330				37	142	2.2×10^2
				32	114	2.0×10^2
				27	123	1.9×10^2
2400				22	117	9.4×10^3
	238	61	TNTC[A]	33	64	7.5×10^3
				34	50	4.8×10^3
0030				32	68	3.1×10^3
	103	34	1.0×10^6	28	83	2.1×10^3
				24	94	2.9×10^5
0100				23	169	1.5×10^5
				22	210	1.1×10^5
	69	12	2.4×10^5	20	218	1.0×10^5
0130				17	207	8.6×10^4
				14	184	7.8×10^4
	127	13	1.3×10^5	10	150	7.6×10^4
0200				8	115	8.8×10^4
				5	84	7.7×10^4
	68	7	4.4×10^6	4	66	7.4×10^4
0230				3	42	8.9×10^4
				2	25	8.9×10^4
	63	8	7.6×10^4	1	14	7.0×10^4
0300				1	9	3.2×10^4
	90	12	4.7×10^3			
0330						
	70	8	2.0×10^3			
0400						

A Too numerous to count

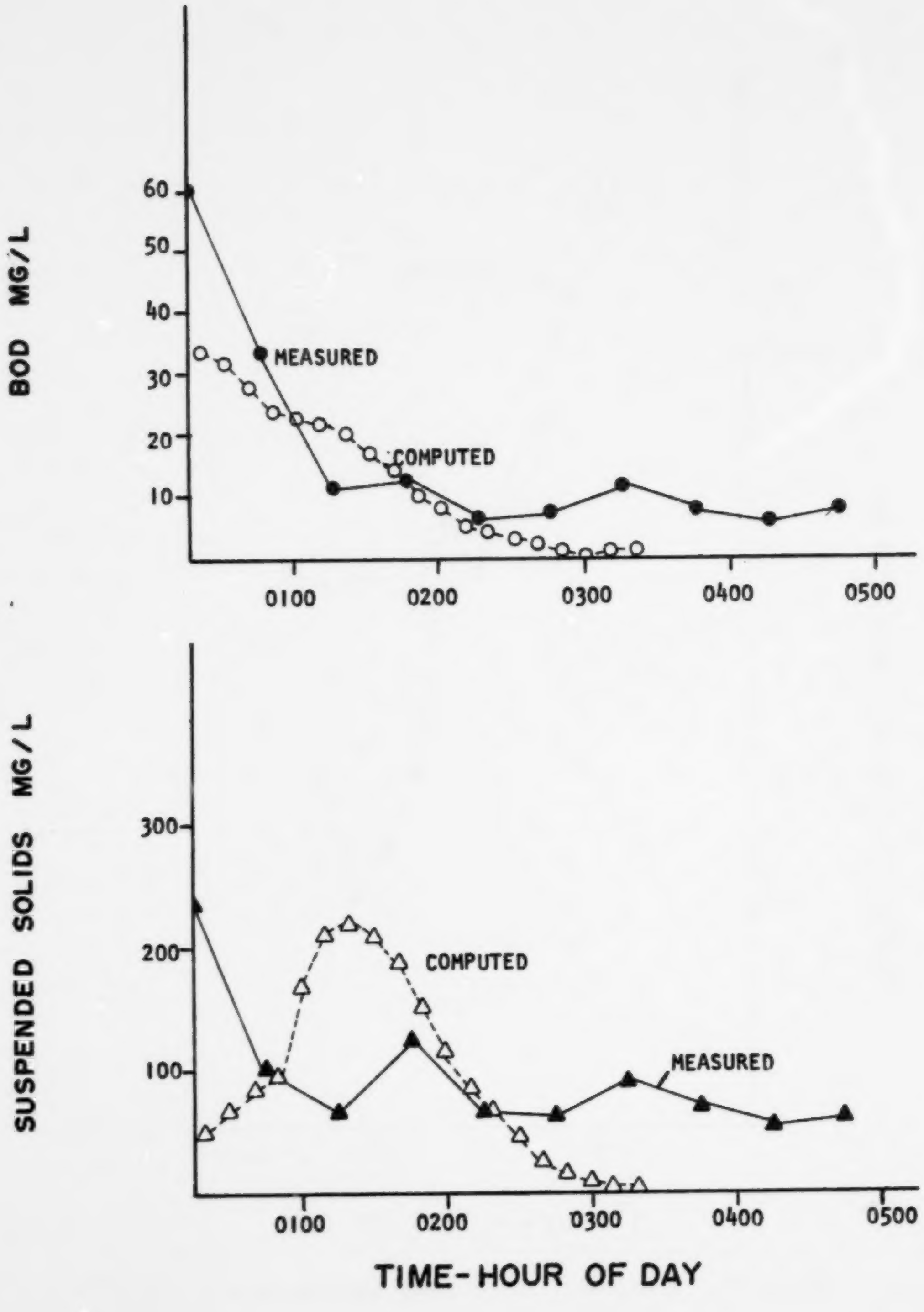


Figure 86. Run Number 26 effluent quality Site I.

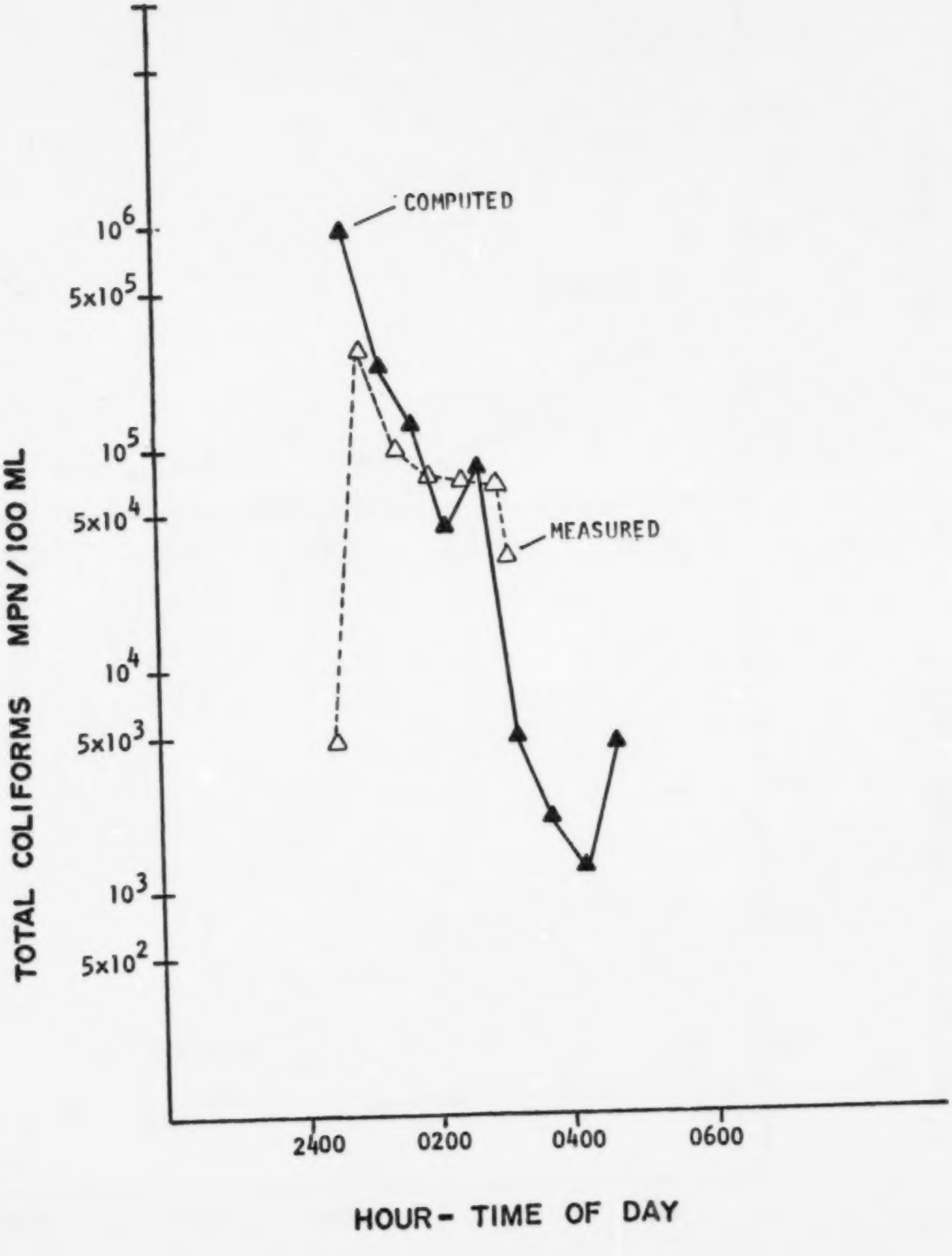


Figure 87. Run Number 26 effluent quality Site I.

TABLE 93. ARRIVING FLOW, SITE II

Run Number 26

Time hours	Arriving flow, cu m/min	Arriving flow, cfs	Computed flow, cu m/min	Computed flow, cfs
2230	0	0	0	0
	0	0	0	0
	0	0	0	0
2300	0	0	0	0
	0	0	0	0
	0	0	0	0
2330	73.3	43.1	2.6	1.5
	80.6	47.4	52.4	30.8
	80.6	47.4	72.1	42.4
2400	73.3	43.1	81.1	47.7
	45.7	26.9	80.0	47.0
	66.1	38.9	75.7	44.5
0030	80.6	47.4	78.7	46.3
	382.7	225.1	119.3	70.2
	382.7	225.1	202.1	118.9
0100	382.7	225.1	274.0	161.2
	382.7	225.1	295.5	173.8
	382.7	225.1	271.7	159.8
0130	382.7	225.1	244.6	143.9
	382.7	225.1	234.1	137.7
	200.8	118.1	326.1	138.9
0200	200.8	118.1	235.8	138.7
	200.8	118.1	224.4	132.0
	200.8	118.1	150.0	88.2
0230	200.8	118.1	106.6	62.7
	102.2	60.1	85.7	50.4
	95.0	55.9	70.6	41.5
0300	80.6	47.4	62.2	36.6
	80.6	47.4	53.9	31.7
	80.6	47.4	41.0	24.1
0330	73.3	43.1	22.3	13.1
0340	66.1	38.9	20.2	11.9
	66.1	38.9	18.0	10.6
0400	66.1	38.9	16.7	9.8
	58.8	34.6	16.0	9.4
	58.8	34.6	14.6	8.6
0430	51.7	30.4	13.6	8.0
	44.4	26.1	13.1	7.7
	37.2	21.9	12.9	7.6
0500	22.8	13.4	12.8	7.5
	18.9	11.1	10.2	6.0
	18.9	11.1	5.1	3.0
0530	18.9	11.1	1.7	1.0
	14.4	8.5		
	7.8	4.6		
0660	1.7	1.0		

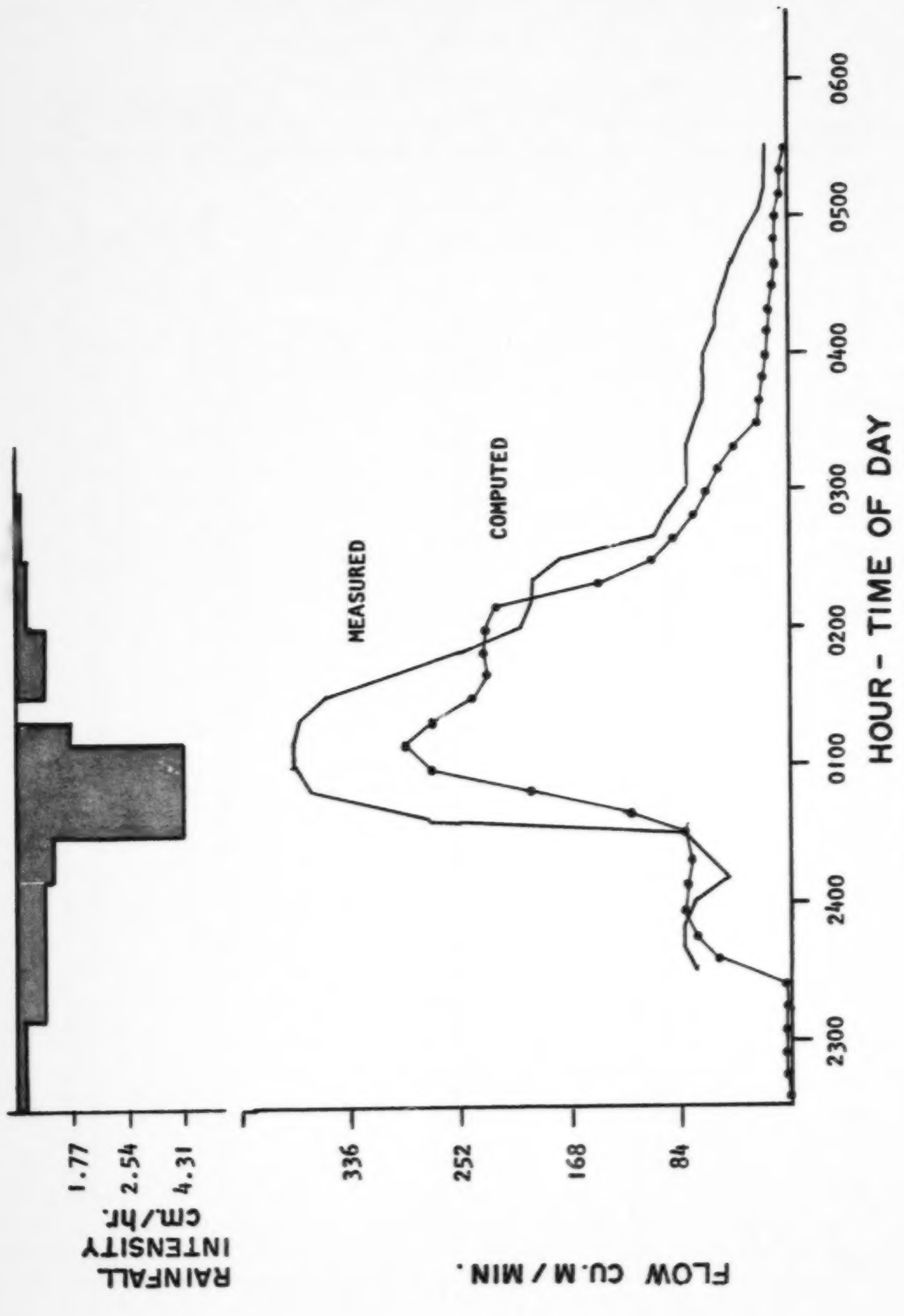


Figure 88. Run Number 26 arriving flow Site 11.

flow to the total computed flow is 0.75. The goodness of fit is excellent.

The arriving quality for run Number 26 is listed in Table 94 and graphically represented in Figures 89 and 90. The measured values show a definite first flush effect at 0100 hours. The measured suspended solids peak at 1538 mg/l while the computed values peak at over 600 mg/l. The computed BOD values are very close to the measured with both showing peaks of about 60 mg/l. The overall comparison of the BOD data is very good while the suspended solids data show only fair correlation because of the large measured peak at 0100 hours. The computed coliform data closely parallels the measured values as shown in Figure 90. The goodness of fit on the quality data is acceptable.

The effluent quality data for Site II are presented in Table 95. Figures 91 and 92 present these data in graphical form. The computed effluent BOD is close to the measured now that the large peak after 0100 hours is dampened. In the arriving quality the computed values were lower and this trend is carried through the Storage block output. The effluent suspended solids pollutograph magnifies the differences of the arriving quality. Thus, the computed suspended solids are generally much lower than the actual as shown in Figure 91. The effluent coliform comparison of Figure 92 shows little correlation which could be the result of over chlorination during the operation of the treatment units. The overall goodness of fit of the quality data is poor.

Run Number 27

The next discretely sampled overflow event was run Number 27 which occurred on May 5, 1974. The total rainfall contributing to Site I was 0.29 centimeters (0.11 inch) and 0.39 centimeters (0.16 inch) to Site II. There were five days in which the cumulative rainfall was less than 2.54 centimeters (1 inch) of rainfall. Table E6, Appendix VI-E lists the rainfall intensities for each site at five minute intervals throughout the run. Because of the small amount of rain that was recorded, the computed flows for Site I did not reach a high enough magnitude to pass over the overflow weirs contributing to Site I. Thus, in Table 96 and Figure 93 no computed flow arrived. Figure 93 presents the measured arriving flow which peaked briefly at 21.9 cu m/min (7.6 cfs). There are two possible reasons for zero computed flow. One is the inaccuracy of the raingages and the other is the difficulty in modeling the flow dividers upstream of Site I (element numbers 37, 209, and 46). Data are available on the characteristics of these structures but field inspection proved this data to be inaccurate. High flows in these structures during dry weather make accurate measurements difficult. The most likely possibility is that the raingauge measurements at such low intensities are inaccurate. The goodness of fit is therefore unacceptable.

During this run discrete samples of the influent and effluent were obtained. Because there was no computed flow for Site I, it was decided to use the quality of the flow above the main flow divider for Site I (Number 37) to

TABLE 94. ARRIVING QUALITY, SITE II

Run Number 26

Time hours	Actual BOD mg/l	Actual SS mg/l	Actual Coliforms No./100 ml	Computed BOD mg/l	Computed SS mg/l	Computed Coliforms MPN/100 ml
2400				71	653	$3.5x10^{7}$
				60	612	$2.6x10^{7}$
				51	516	$1.7x10^{7}$
0030	32	427	$6.6x10^{6}$	42	418	$8.3x10^{6}$
				34	360	$2.8x10^{6}$
	58	1538	$8.3x10^{6}$	23	280	$1.5x10^{6}$
0100				14	182	$8.3x10^{5}$
	20	677	$1.4x10^{6}$	13	197	$2.7x10^{5}$
				12	198	$1.9x10^{5}$
0130	16	473	$5.8x10^{5}$	12	199	$1.7x10^{5}$
				11	193	$1.7x10^{5}$
				10	172	$1.8x10^{5}$
0200	21	457	$3.4x10^{5}$	8	152	$1.7x10^{5}$
				7	130	$2.3x10^{5}$
				6	113	$2.7x10^{5}$
0230	14	298	$1.1x10^{6}$	5	102	$3.0x10^{5}$
				4	81	$3.2x10^{5}$
				3	61	$3.4x10^{5}$
0300	16	242	$9.5x10^{5}$	3	44	$4.0x10^{5}$
				2	35	$5.1x10^{5}$
				2	28	$5.9x10^{5}$
0330				2	17	$7.5x10^{5}$
				2	15	$1.0x10^{5}$
				2	15	$1.3x10^{5}$
0400						
	5	64	$8.5x10^{5}$			
0430						

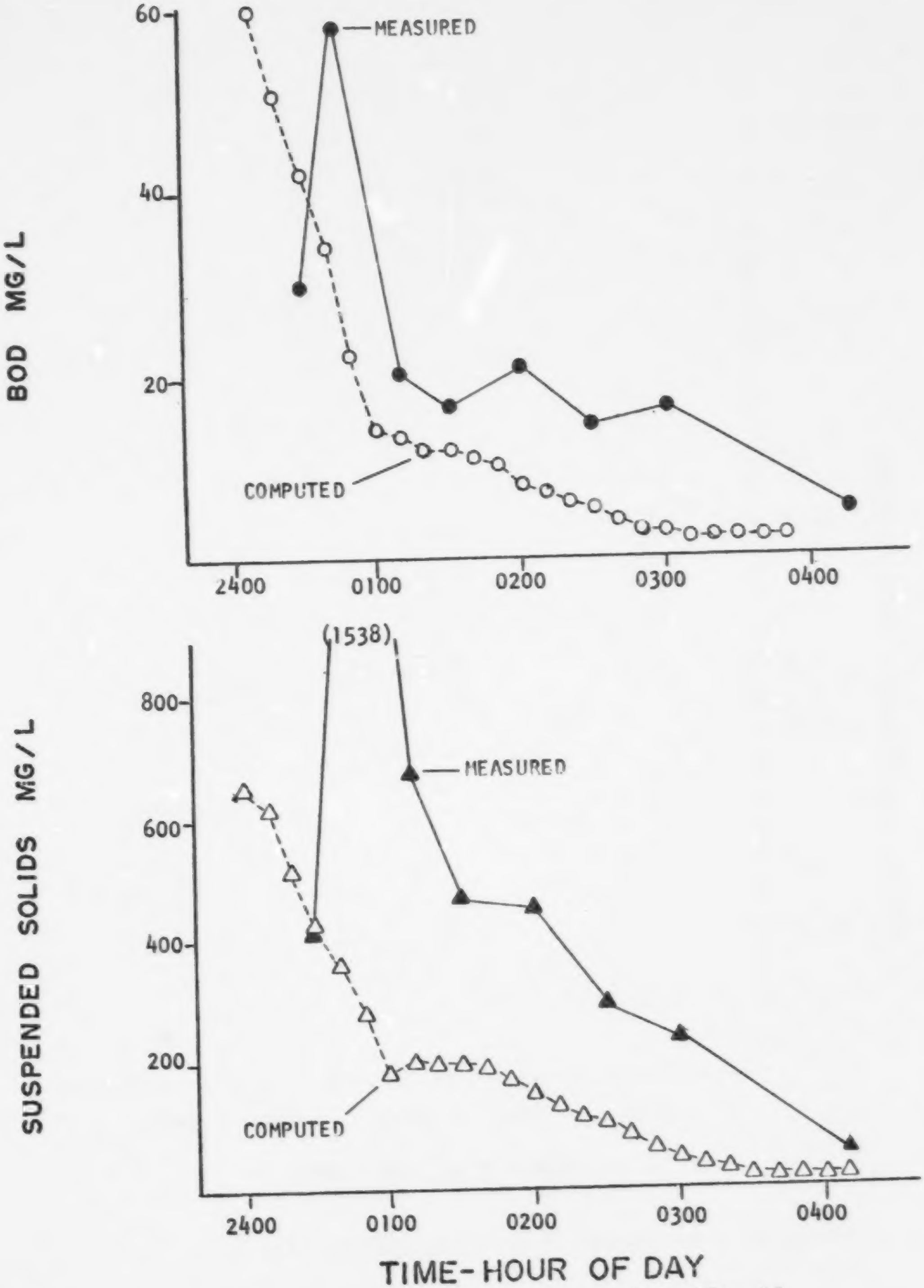


Figure 89. Run Number 26 arriving quality Site 11.

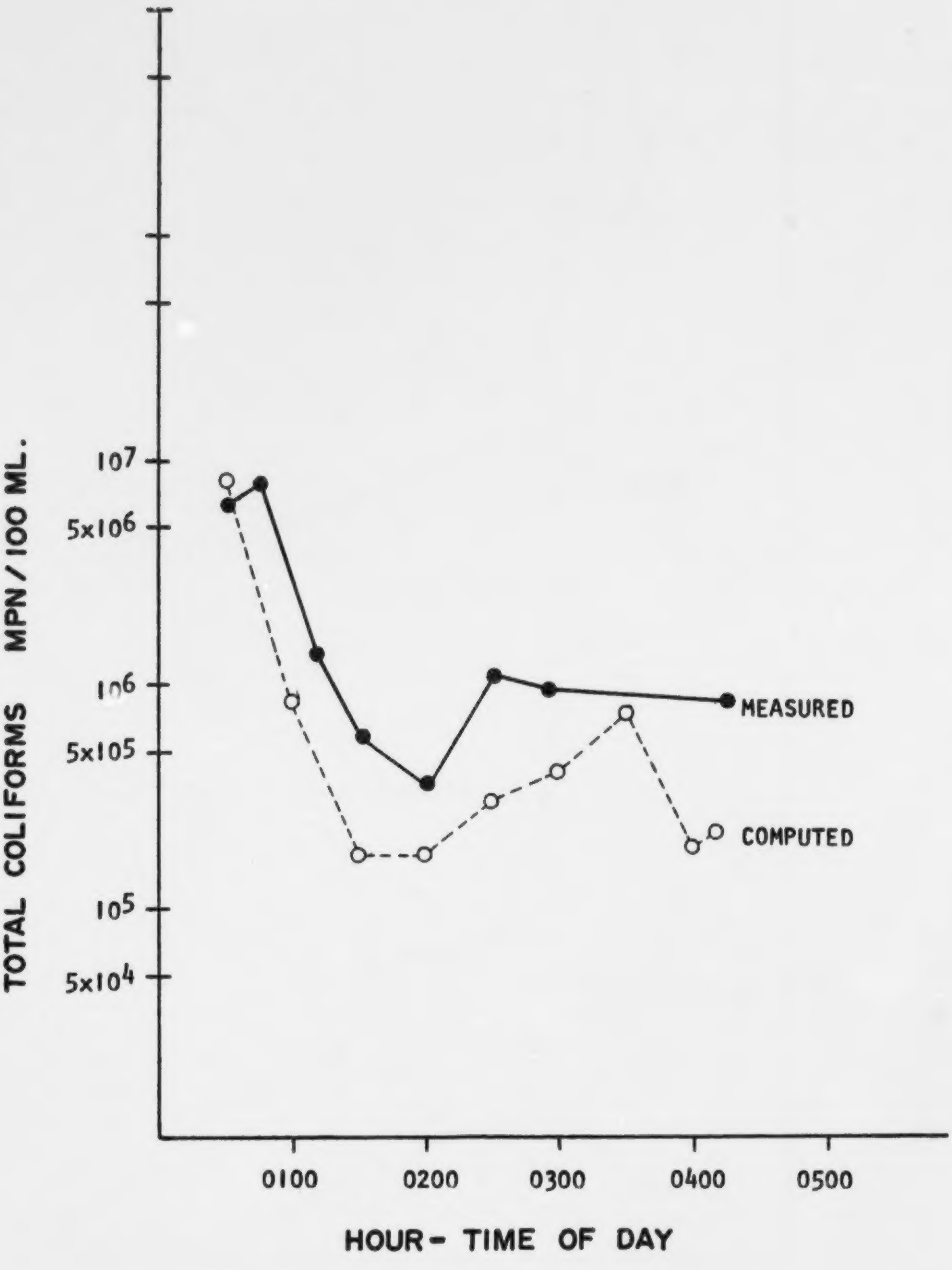


Figure 90. Run Number 26 arriving quality Site II.

TABLE 95. EFFLUENT QUALITY, SITE II

Run Number 26

	Actual			Computed		
Time hours	BOD mg/l	SS mg/l	Coliforms No./100 ml	BOD mg/l	SS mg/l	Coliforms MPN/100 ml
0030	12	62		16	72	1.3×10^3
				11	84	7.1×10^2
	9	93		10	125	9.9×10^4
0100				10	127	1.1×10^5
	17	493		10	147	9.6×10^4
				9	137	8.6×10^4
0130	9	321		8	131	7.9×10^4
				7	117	7.2×10^4
				6	104	7.1×10^4
0200	10	479	<9[A]	5	91	6.7×10^4
				4	74	4.6×10^4
				3	41	1.2×10^2
0230	7	279	8.2×10^4	2	23	1.6×10^2
				2	14	1.9×10^2
	6	160	6.5×10^2	1	8	2.2×10^2
0300				1	7	2.5×10^2
	3	94	30[B]	1	7	
				1	6	
0330	5	92	40[B]			
0400						
	2	41	<9[A]			
0430						

A. No growth
B. Less than 20 colonies

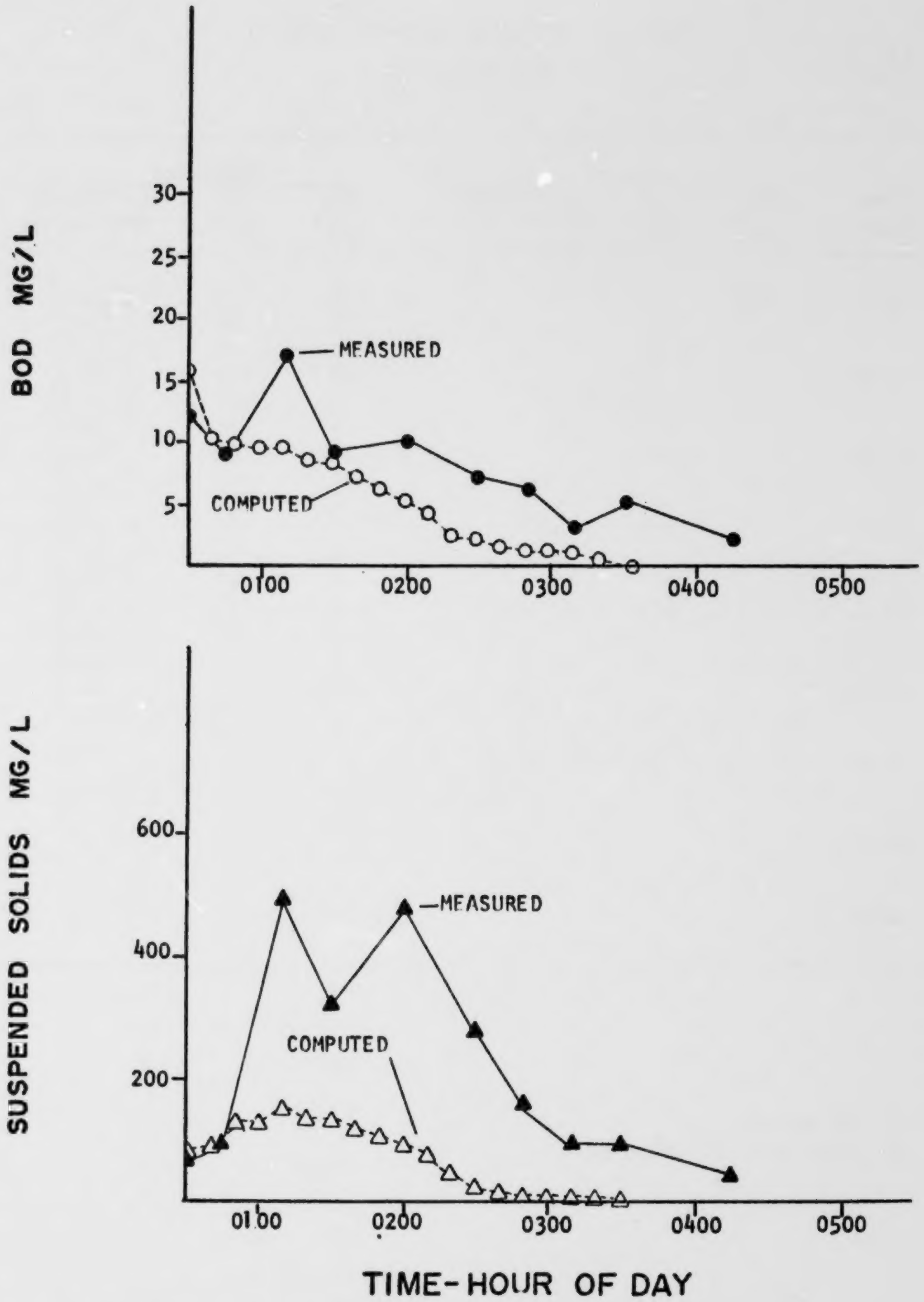


Figure 91. Run Number 26 effluent quality Site 11.

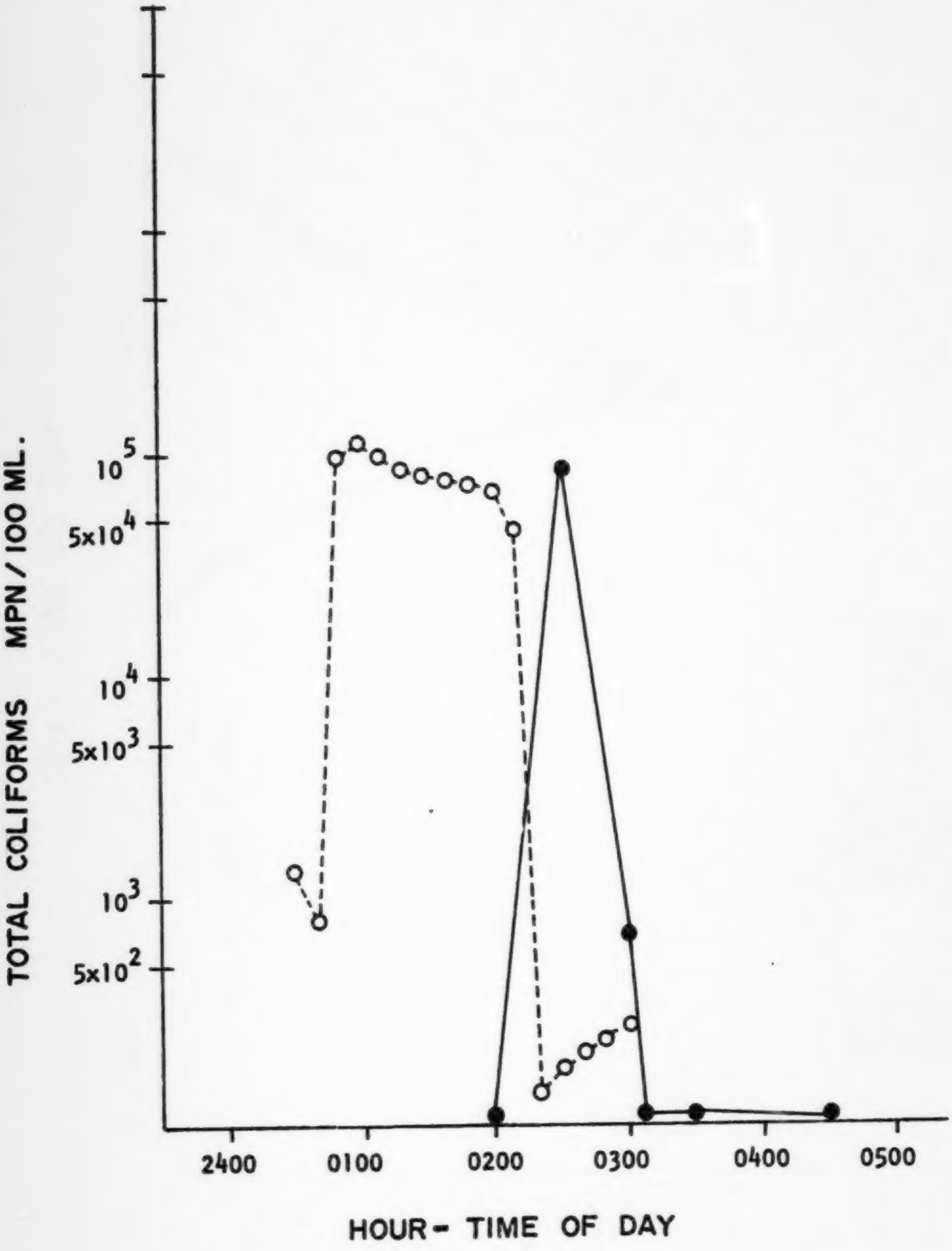


Figure 92. Run Number 26 effluent quality Site II.

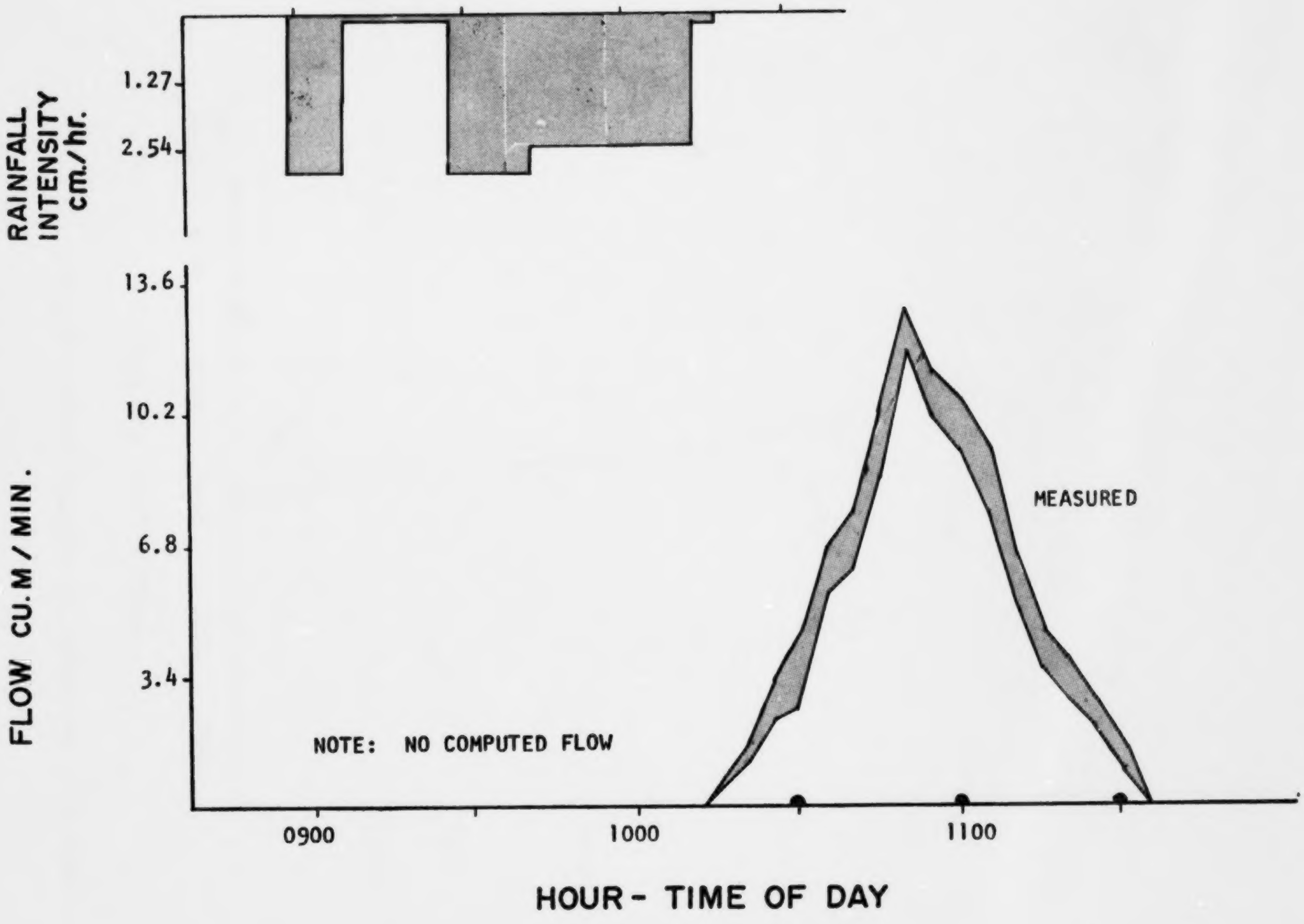


Figure 93. Run Number 27 arriving flow Site 1.

TABLE 96. ARRIVING FLOW, SITE I

Run Number 27

Time hours	Arriving flow,				Computed flow,	
	cu m/min		cfs			
	min.	max.	min.	max.	cu m/min.	cfs
1015	0	0	0	0	0	0
	1.4	2.4	0.6	0.8	0	0
	3.2	5.4	1.3	1.9	0	0
1030	4.6	7.8	1.5	2.7	0	0
	6.6	11.2	3.2	3.9	0	0
	7.7	13.0	3.6	4.5	0	0
1045	10.2	17.3	5.0	6.0	0	0
	12.9	21.9	6.9	7.6	0	0
	11.2	19.0	6.0	6.6	0	0
1100	10.5	17.8	5.4	6.2	0	0
	9.2	15.6	4.5	5.4	0	0
	5.3	9.0	3.1	3.9	0	0
1115	4.6	7.8	2.1	2.7	0	0
	3.9	6.6	1.7	2.3	0	0
	2.7	4.6	1.3	1.6	0	0
1130	1.5	2.5	0.7	0.9	0	0
	0.0	0.0	0	0	0	0

compare with the actual measured data. This procedure is acceptable because the flows in excess of the flow divider are normally transported this short distance to the Site I wetwell without any other inflows. This procedure allows full use of the available discrete data. Table 97 lists the actual and computed quality for this run while Figures 94 and 95 present a graphical comparison of the data. The upstream computed BOD remains relatively constant at 100 mg/l which is near the dry weather flow average while the measured values decline from 100 mg/l to less than 25 mg/l in the hour of sampling. The suspended solids comparison is very close throughout the duration of the run both in magnitude and trend. The coliform data of Figure 95 are close in magnitude, but the measured values decrease after the initial peak is reached while the computed values remain relatively constant. The goodness of fit of the quality data is acceptable.

The problem with zero computed flow at Site I was not found for the flows at Site II as listed in Table 98 and graphed in Figure 96. The measured flow peaks above 22 cubic meters per minute (13 cfs), because during this small rainfall event the sluice gate was down for the duration of the run. This negated the dampening effects of the flow divider (element No. 87). Generally, the computed flow is much higher than the measured flow throughout the duration of the run. The overall predictive capacity of the SWMM and goodness of fit is only fair for this run. The ratio of the total actual flow to the computed flow is 0.54.

Table 99 lists the quality of the arriving flow for this run at Site II. Figures 97 and 98 present the graphical comparison of the measured and computed data. The BOD data shows only fair correlation and the computed remains relatively constant at 70 mg/l. The suspended solids data correlate very well and the computed values follow the same trend as the measured. The coliform data are within the same range but the computed values do not follow the measured trend. The goodness of fit of the arriving quality data is acceptable.

Table 100 lists the measured and computed values of effluent quality at Site II during run 27. Figures 99 and 100 present the graphical comparison of this data. The BOD data is now close in magnitude because the peaks of the arriving pollutograph are dampened through the treatment units. The suspended solids data show that the Storage block removed more suspended solids than was actually measured. This removal might be due to the low concentration of the measured arriving flows. The coliform data show good correlation. The goodness of fit of the effluent quality data is acceptable based on the difference of the influent concentrations.

Run Number 30

The fifth discretely sampled run was Number 30 which occurred on May 13, 1974. The average total rainfall for the area was 0.59 centimeters (0.23 inches) with a duration of 60 minutes. There were two days prior to this run when no rain fell and six days when the cumulative rainfall was less than 2.54 centimeters (1 inch). Table E7, Appendix VI-E,

TABLE 97. ARRIVING QUALITY, SITE 1

Run Number 27

Time hours	Actual BOD mg/l	Actual SS mg/l	Actual Coliforms No./100 ml	Upstream Computed BOD mg/l	Upstream Computed SS mg/l	Upstream Computed Coliforms MPN/100 ml
1000				113	223	3.5×10^7
				106	224	2.7×10^7
				104	275	2.3×10^7
1015				104	306	2.3×10^7
				103	317	2.2×10^7
				103	300	2.3×10^7
1030	114	261	1.0×10^7	106	256	2.6×10^7
				109	199	2.9×10^7
	114	215	2.1×10^7	115	158	3.2×10^7
1045				119	132	3.1×10^7
	86	201	8.5×10^6	120	117	2.9×10^7
				121	108	2.7×10^7
1100	54	121	7.5×10^6	120	100	2.5×10^7
				119	94	2.3×10^7
	38	81	2.7×10^6	117	89	2.1×10^7
1115				115	84	2.0×10^7
	31	66	1.8×10^6	112	80	1.8×10^7
				110	76	1.7×10^7
1130	29	42	1.7×10^6	108	73	1.6×10^7
				106	69	1.6×10^7
	17	60	1.4×10^6	104	66	1.5×10^7
1145				102	64	1.4×10^7
				100	61	1.2×10^7

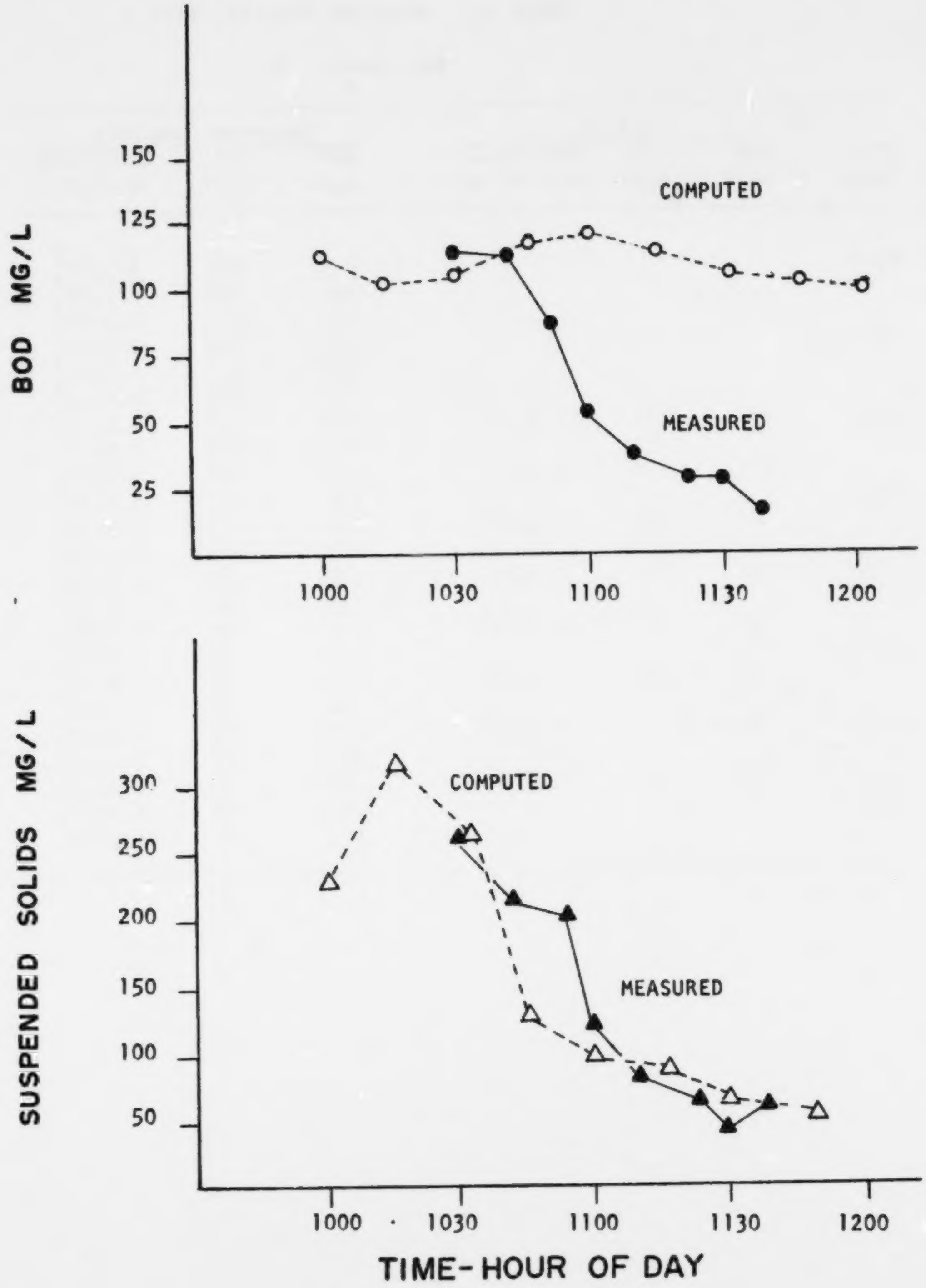


Figure 94. Run Number 27 arriving quality Site I.

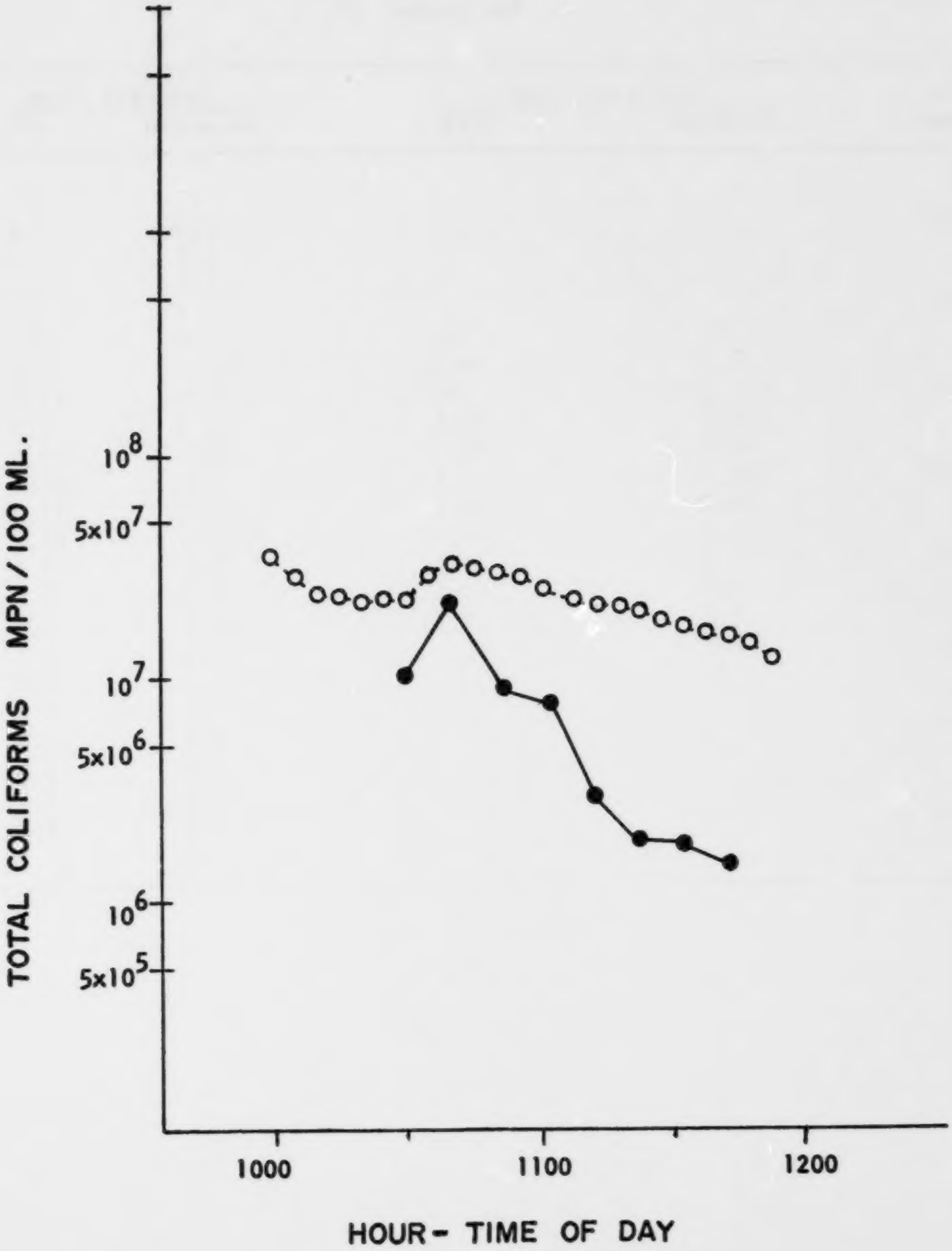


Figure 95. Run Number 27 arriving quality, Site I.

TABLE 98. ARRIVING FLOW, SITE II

Run Number 27

Time	Arriving flow,		Computed flow,	
hours	cu m/min	cfs	cu m/min	cfs
			0	0
0900			3.0	1.8
			3.5	2.1
			4.7	2.8
0930			6.3	3.7
	0	0	8.6	5.1
	0.8	0.5	11.6	6.8
1000	1.7	1.0	20.2	11.9
	22.7	13.4	32.3	19.0
	19.0	11.2	38.4	22.6
1030	22.7	13.4	35.5	20.9
	19.0	11.2	27.0	15.9
	17.0	10.0	21.0	12.4
1100	7.6	4.5	16.6	9.8
	5.6	3.3	15.0	8.8
	5.6	3.3	13.4	7.9
1130	1.8	1.1	12.0	7.1
	0	0	11.2	6.6
			10.3	6.1
1200			9.7	5.7
			9.1	5.4
			8.8	5.2
1230			8.3	4.9

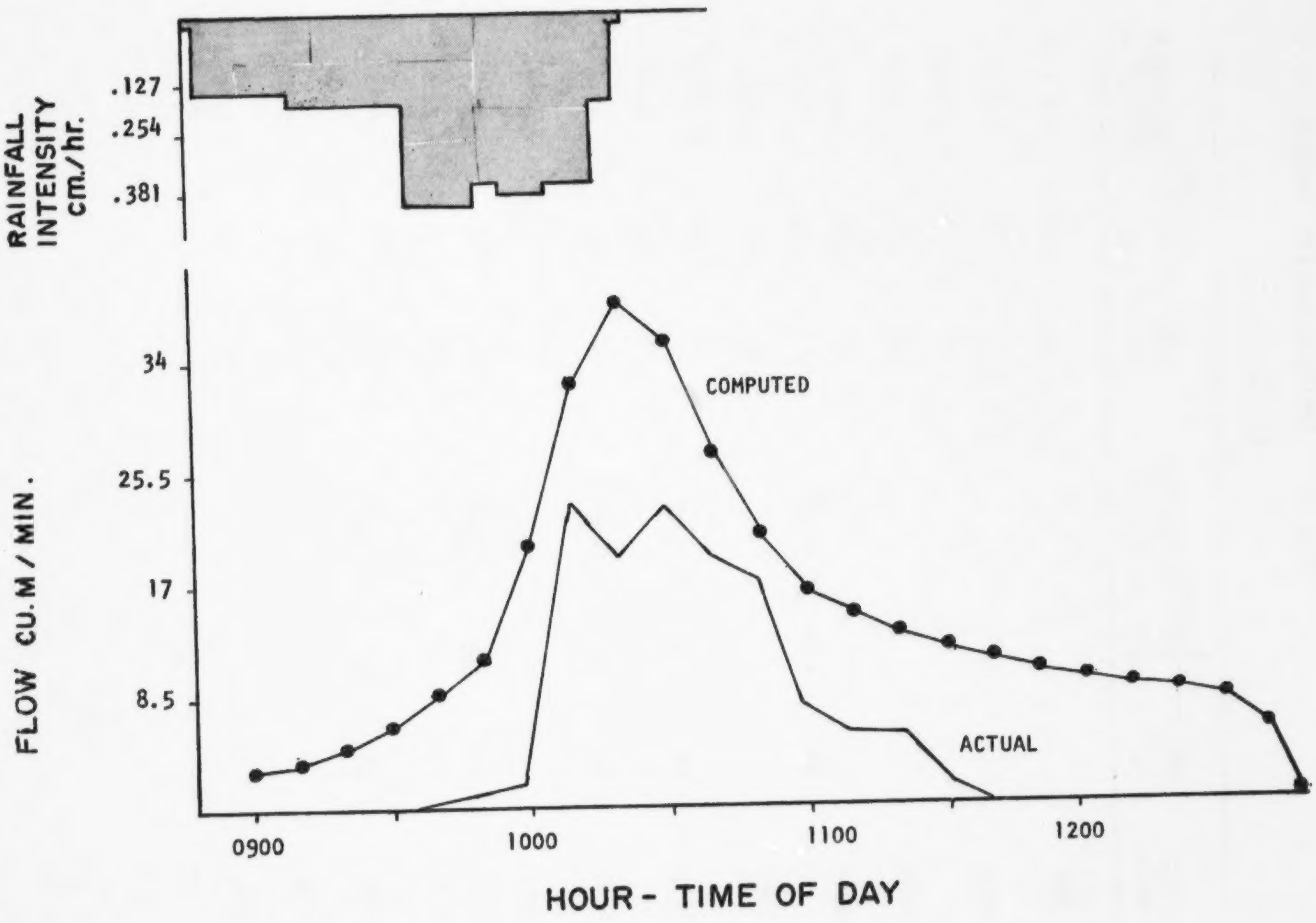


Figure 96. Run Number 27 arriving flow Site 11.

TABLE 99. ARRIVING QUALITY, SITE II

Run Number 27

Time hours	Actual BOD mg/l	Actual SS mg/l	Actual Coliforms No./100 ml	Computed BOD mg/l	Computed SS mg/l	Computed Coliforms MPN/100 ml
0900				64	1108	2.8×10^7
				63	1320	2.6×10^7
0915						
				68	1080	2.4×10^7
0930				69	634	2.0×10^7
0945				67	300	1.6×10^7
	80	412	2.0×10^6	71	172	1.3×10^7
1000				72	142	9.1×10^6
				75	198	5.7×10^6
1015	33	105	8.1×10^7			
				79	268	3.7×10^6
1030	43	151	4.4×10^7	78	337	3.3×10^6
				76	353	3.9×10^6
1045						
				75	289	7.4×10^6
	77	148	3.4×10^7			
1100				77	212	1.0×10^7
				80	148	1.2×10^7
1115	87	64	1.3×10^7			
				80	106	1.3×10^7
1130				79	81	1.3×10^7
1145				77	66	1.3×10^7

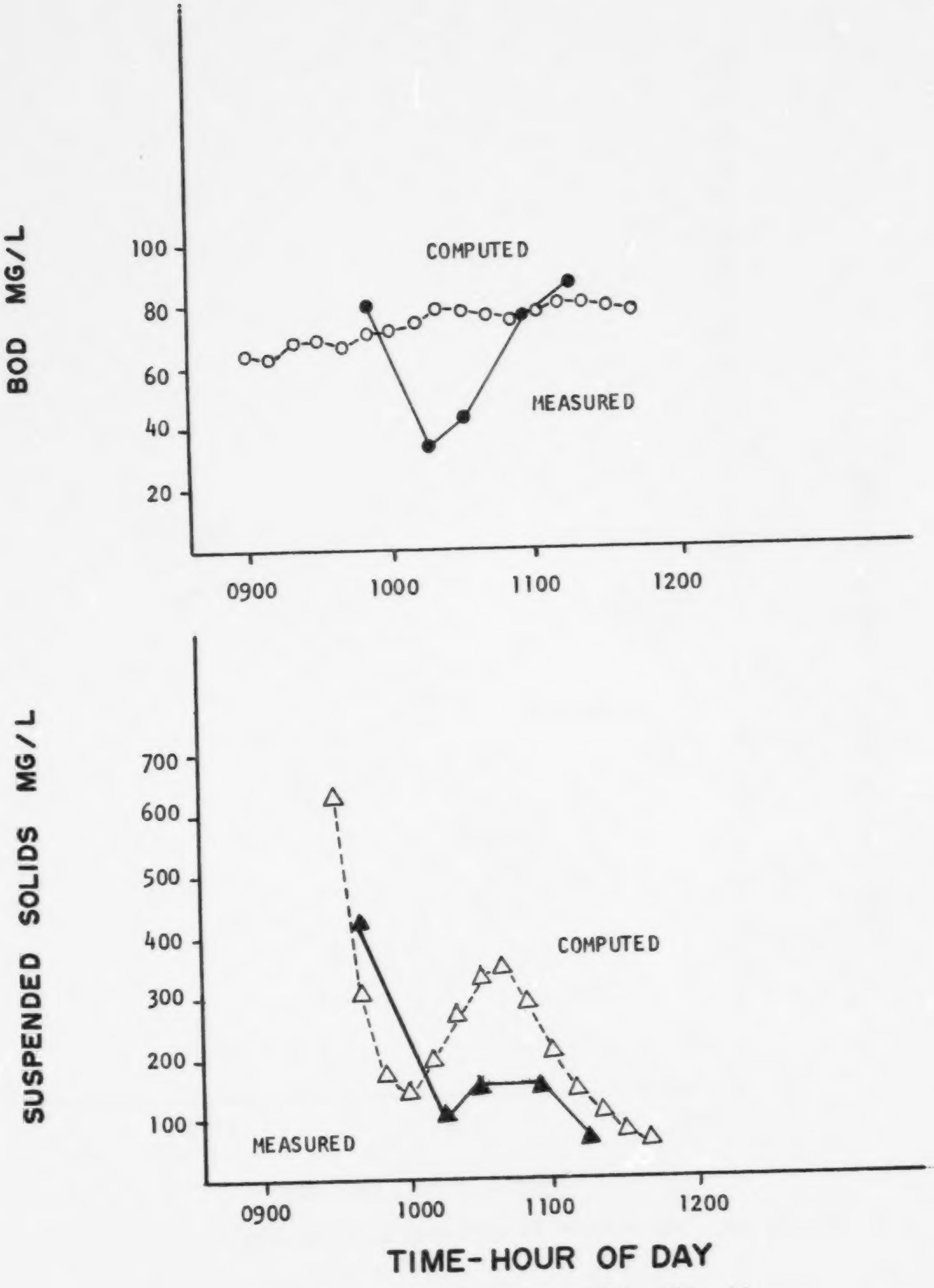


Figure 97. Run Number 27 arriving quality Site II.

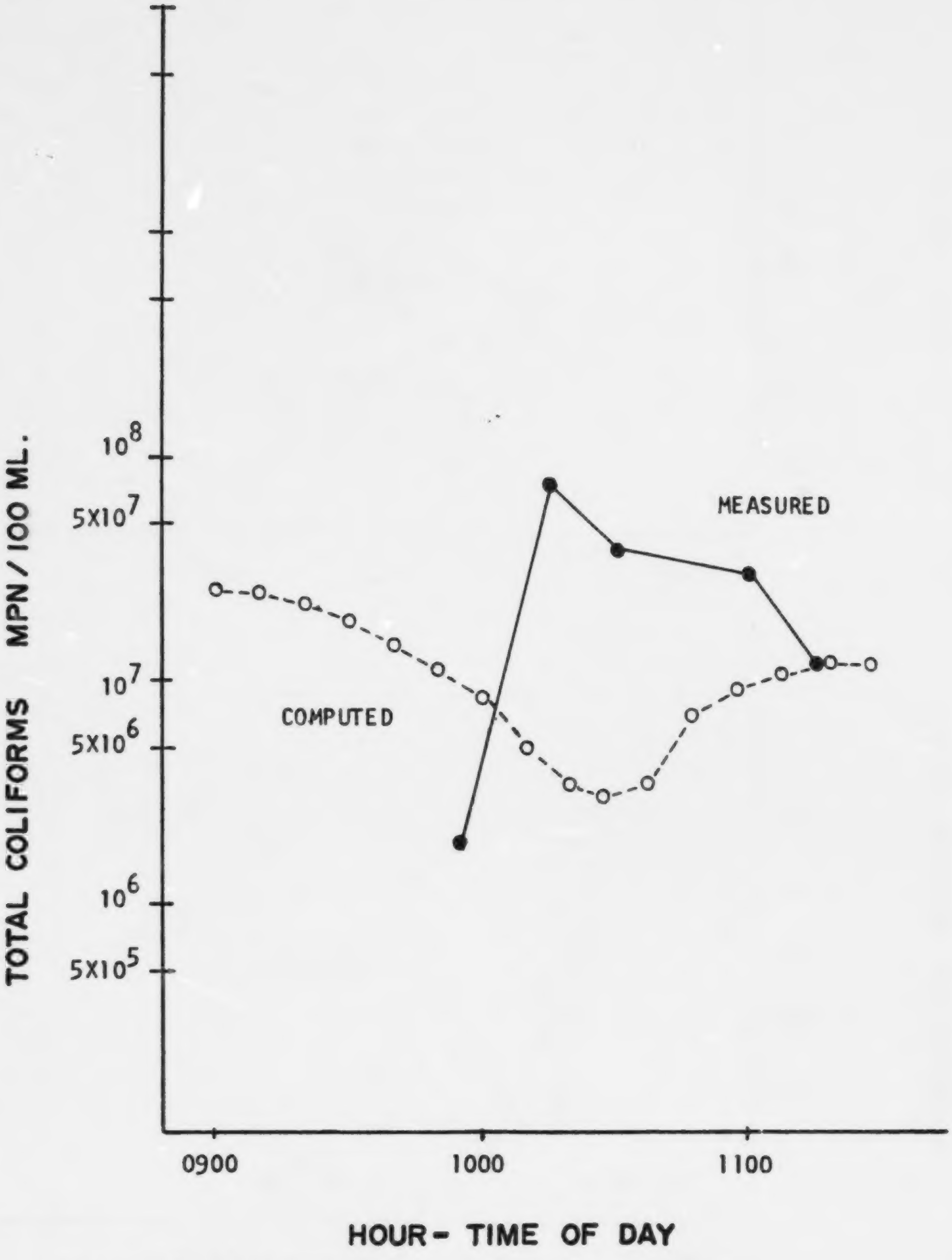


Figure 98. Run Number 27 arriving quality Site II.

TABLE 100. EFFLUENT QUALITY, SITE II

Run Number 27

Time hours	Actual BOD mg/l	Actual SS mg/l	Actual Coliforms No./100 ml	Computed BOD mg/l	Computed SS mg/l	Computed Coliforms MPN/100 ml
1000				36	140	6.7×10^3
				40	92	6.0×10^3
				36	65	5.2×10^3
1015	39	109	8.0×10^3	36	48	4.2×10^3
				32	41	4.2×10^3
				32	49	3.6×10^3
1030	40	99	1.2×10^4	32	56	3.2×10^3
				31	62	3.1×10^3
				31	64	3.2×10^3
1045	37	99	2.8×10^4	30	64	3.9×10^3
				28	64	4.8×10^3
				28	60	6.2×10^3
1100	36	87	2.2×10^4	27	61	8.2×10^3
				26	57	2.6×10^4
				26	57	2.6×10^4
1115	32	75	8.2×10^4	26	56	2.7×10^4
				24	54	2.4×10^4
1130						

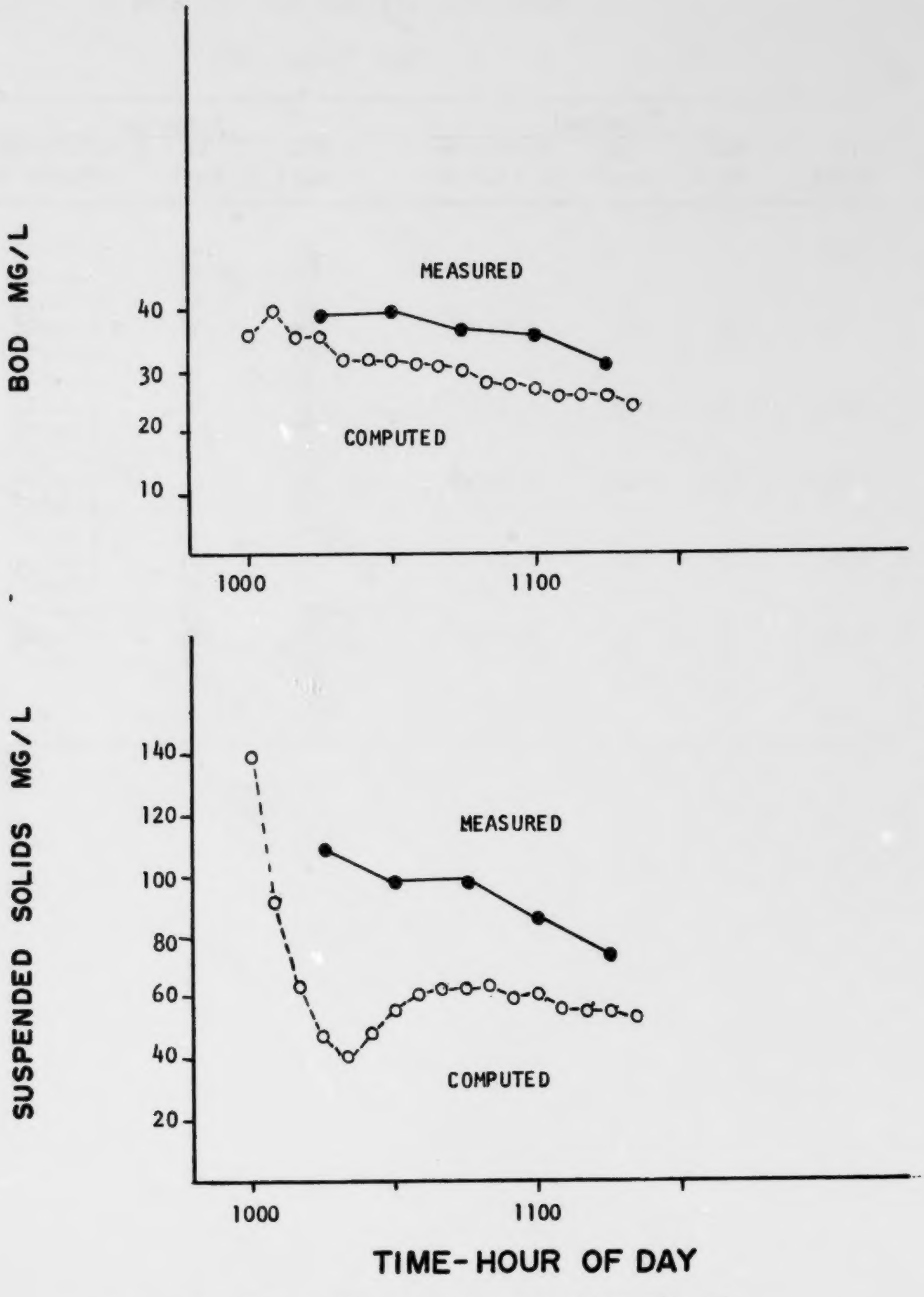


Figure 99. Run Number 27 effluent quality Site II.

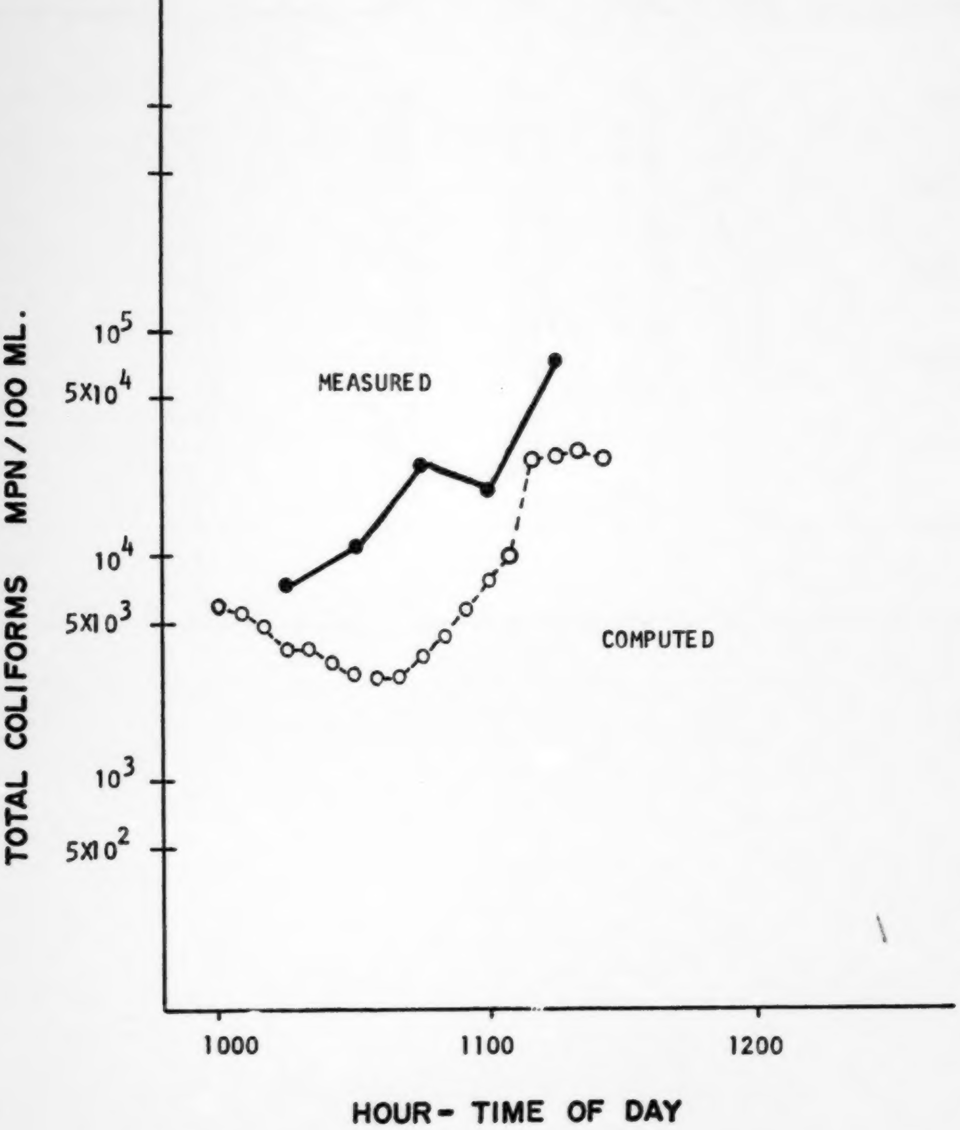


Figure 100. Run Number 27 effluent quality Site II.

TABLE 101. ARRIVING FLOW, SITE I

Run Number 30

Time hours	Arriving flow, cu m/min min	Arriving flow, cu m/min max.	Arriving flow, cfs min	Arriving flow, cfs max.	Computed flow, cu m/min	Computed flow, cfs
1200	0.0	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0
1230	0.0	0.0	0.0	0.0	1.7	1.0
	17.3	25.7	10.2	15.1	9.5	5.6
	21.9	28.1	12.9	16.5	8.8	5.2
1300	28.7	34.0	16.9	20.0	6.8	4.0
	31.8	36.4	18.7	21.4	20.7	12.2
	31.8	35.7	18.7	21.0	29.4	17.3
1330	20.4	25.7	12.0	15.1	26.0	15.3
	12.1	16.7	7.1	9.8	31.5	18.5
	9.0	13.6	5.3	8.0	28.4	16.7
1400	5.8	9.9	3.4	5.8	26.0	15.3
	4.4	8.3	2.6	4.9	27.0	15.9
	3.9	7.8	2.3	4.6	19.7	11.6
1430	3.1	6.8	1.8	4.0	9.0	5.3
					3.4	2.0
					0.0	0.0

lists the rainfall intensities for each site at 5-minute intervals throughout the run. Table 101 lists the computed and measured values of the arriving flows to Site I. Figure 101 presents a graphical comparison of the flows which shows the computed flows to lag behind the measured by about 30 minutes. The ratio of the actual to the computed total flow flow arriving is 1.05. Because of the time difference between the peaks, the closeness of fit of the two hydrographs is acceptable, but could be very good without the lag.

The arriving quality of the flow at Site I was determined by 10 samples of the flow taken at 10-minute intervals throughout the run. Table 102 lists the actual and computed quality values and Figures 102 and 103 present the graphical comparison. The BOD values are close throughout the run although the "first flush" value from the measured graph is much higher than the computed. The suspended solids data are close throughout the duration and the initial computed and measured values are very close. The coliform comparison in Figure 103 are generally close except for the large drop in the actual values at 1330 hours. Although the coliform pollutograph may appear to be out of phase because of these drops, such is not the case because the other pollutographs for this site are in phase. The goodness of fit of the quality data is acceptable.

The effluent quality from the treatment unit at Site I and the computed effluent quality are listed in Table 103 and graphically compared in Figures 104 and 105. The BOD comparison maintains the trend shown in the arriving quality, in that the actual values are higher in concentration than the computed. The initial high peak shown by the measured values at 1300 hours relates to the first flush concentration of the influent and the fact that poor removals are experienced from the treatment units during the first few minutes of operation. The Storage block has simulated the BOD removals relatively well for the remaining values. The effluent suspended solids comparison is very close throughout the run. The high concentrations of the influent values (700 mg/l) are reduced to 150 mg/l range at the start of the effluent values. The remainder of the run is very close showing good correlation between the computed and actual values. The coliform comparison shown in Figure 105 shows little similarity between the computed and the actual. But the low concentrations shown at the start of the computed values may be neglected since the arriving computed flow is very low at this same time. When the computed flow to the treatment units increases, the concentrations stabilize at about 10^4 MPN/100 ml. The actual coliform numbers show the opposite tendency. Initially the measured values are very high and then stabilize. The high values may be traced to poor chlorination at the start of the treatment unit operation or from the discharge of the flotation tanks to the effluent channel of the initial non-chlorinated flows. After these modifications are taken into consideration, the results maintain the small differences shown in the arriving quality and the overall goodness of fit is acceptable.

The actual and computed flows arriving at Site II during this run are

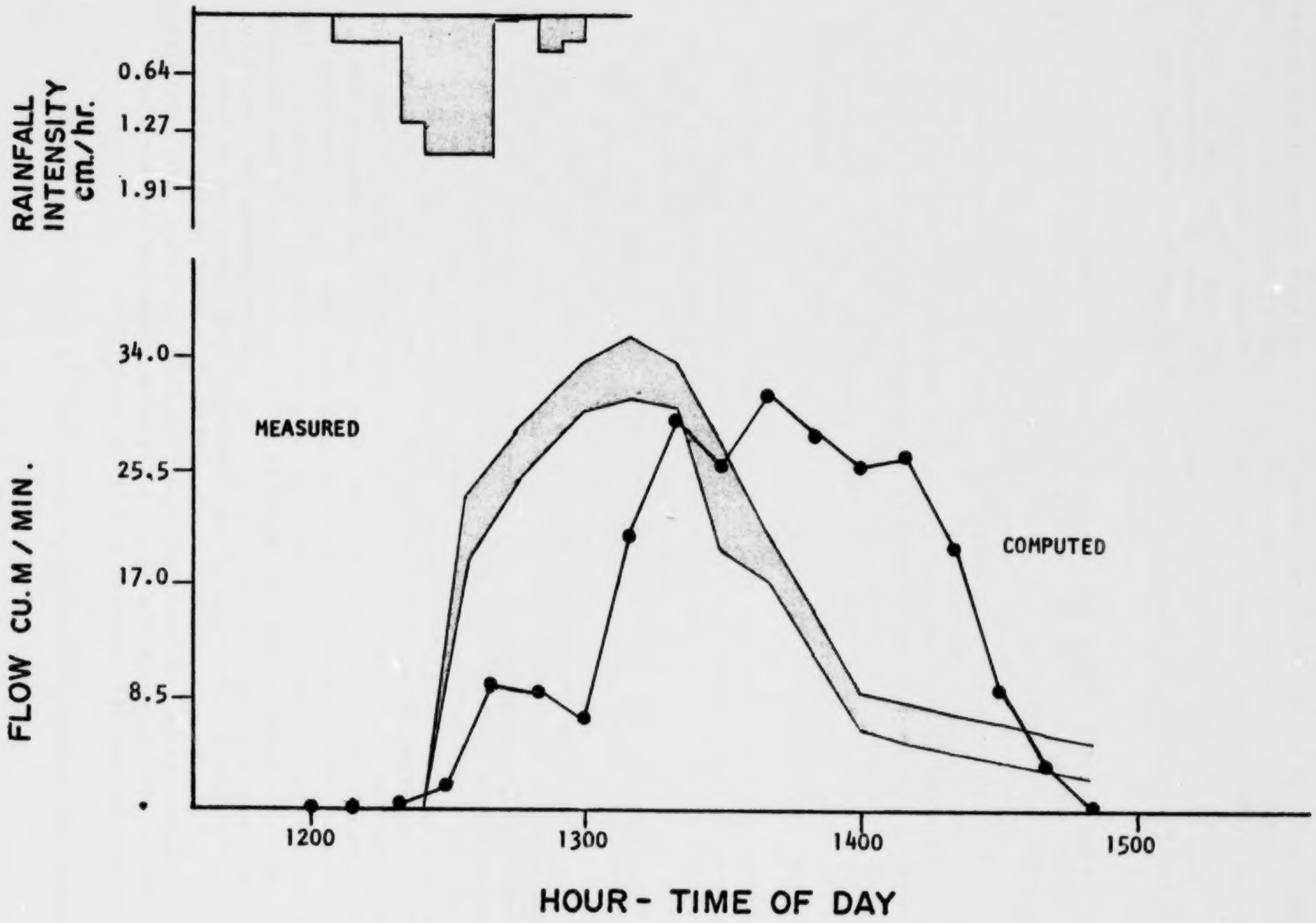


Figure 101. Run Number 30 arriving flow Site 1.

TABLE 102. ARRIVING QUALITY, SITE 1

Run Number 30

Time hours	Actual BOD mg/l	Actual SS mg/l	Actual Coliforms No./100 ml	Computed BOD mg/l	Computed SS mg/l	Computed Coliforms MPN/100 ml
1230				137	347	5.0×10^7
	>225	725	2.0×10^7	104	726	1.2×10^7
	137	458	1.4×10^7	96	1179	4.1×10^5
1300	145	400	6.4×10^6	85	681	1.3×10^7
	111	340	6.5×10^6	99	353	1.8×10^7
	119	390	3.1×10^6	95	355	1.5×10^7
1330	103	375	5.0×10^4	72	308	9.6×10^6
	72	214	2.5×10^6	76	358	8.1×10^6
	66	181	1.2×10^6	69	343	5.0×10^6
1400	66	146	2.8×10^6	62	319	3.9×10^6
	67	127	5.0×10^6	56	291	3.5×10^6
				64	123	2.0×10^6
1430				62	81	1.6×10^6
				60	43	1.4×10^6
1500						

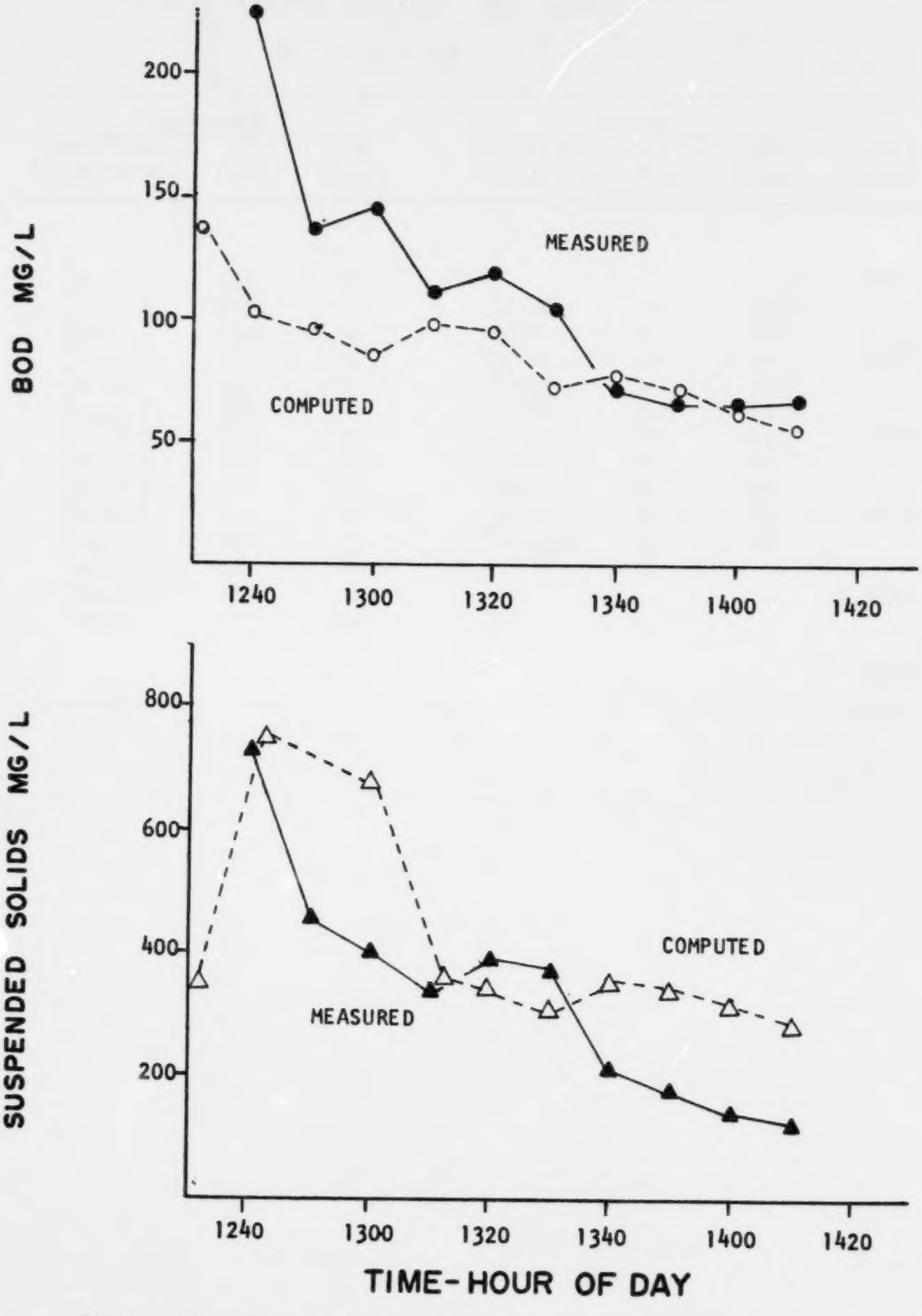


Figure 102. Run Number 30 arriving quality Site 1.

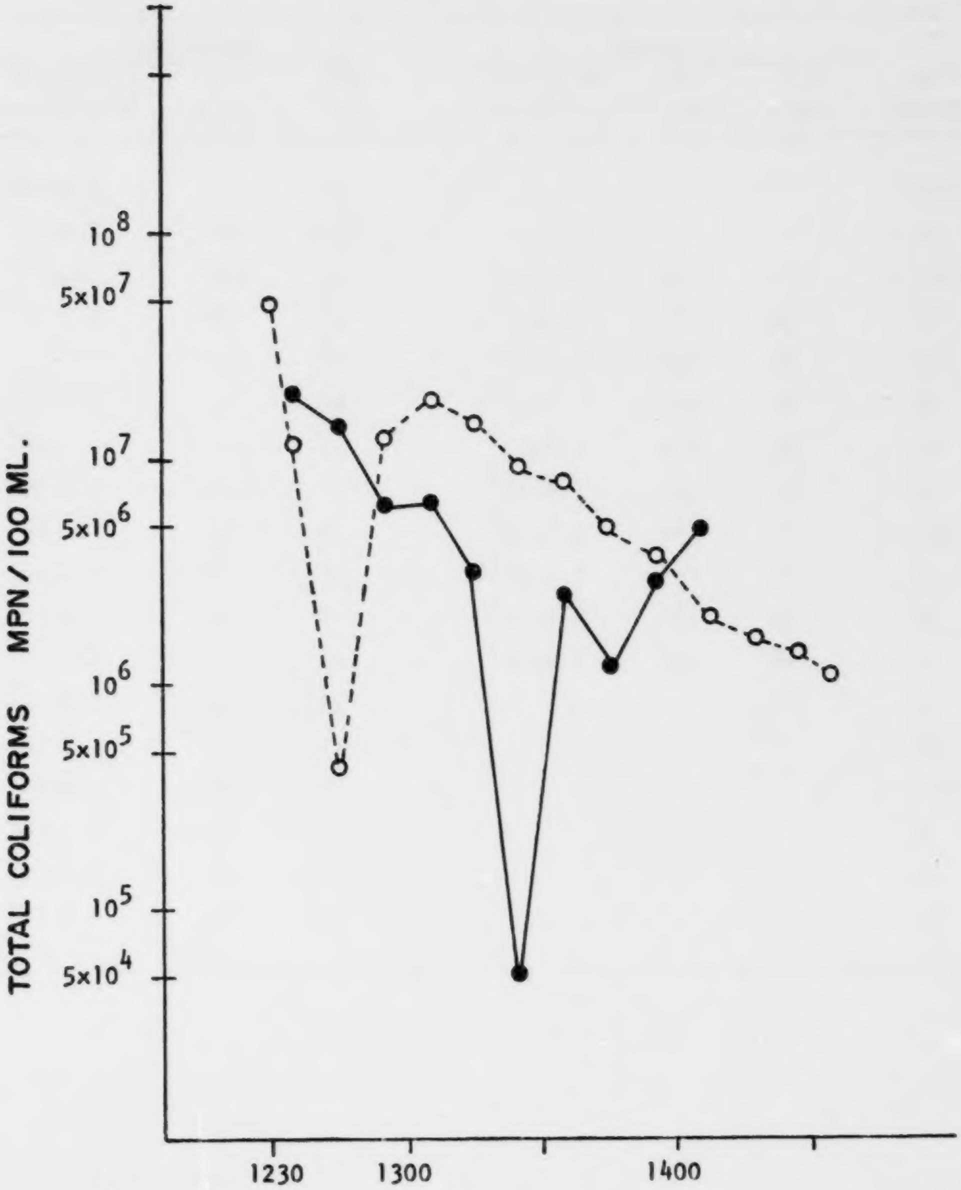


Figure 103. Run Number 30 arriving quality Site I.

TABLE 103. EFFLUENT QUALITY, SITE I

Run Number 30

	Actual			Computed		
Time hours	BOD mg/l	SS mg/l	Coliforms No./100 ml	BOD mg/l	SS mg/l	Coliforms MPN/100 ml
1240				31	71	$3.9x10^1$
50				36	128	$4.0x10^2$
1300	144	58	$8.9x10^5$	35	160	$2.6x10^2$
10	96	263	$1.5x10^4$	28	138	$2.7x10^2$
20	90	259	$4.2x10^3$	--	--	--
30	62	195	$2.0x10^3$	44	45	$2.1x10^4$
40	55	171	$7.7x10^4$	41	44	$1.3x10^4$
50	51	164	$1.3x10^4$	37	43	$9.4x10^3$
1400	59	144	$3.3x10^3$	34	41	$7.1x10^3$
10	53	136	$1.1x10^3$	31	38	$5.6x10^3$
20	50	126	$1.1x10^3$	28	35	$4.6x10^3$
30	48	105	$1.2x10^3$	26	31	$4.0x10^3$
40				23	27	$3.6x10^3$
50				21	26	$3.2x10^3$
1500				18	24	$2.8x10^3$
10				17	23	$2.5x10^3$
20				16	20	$2.3x10^3$
30				15	19	$2.1x10^3$

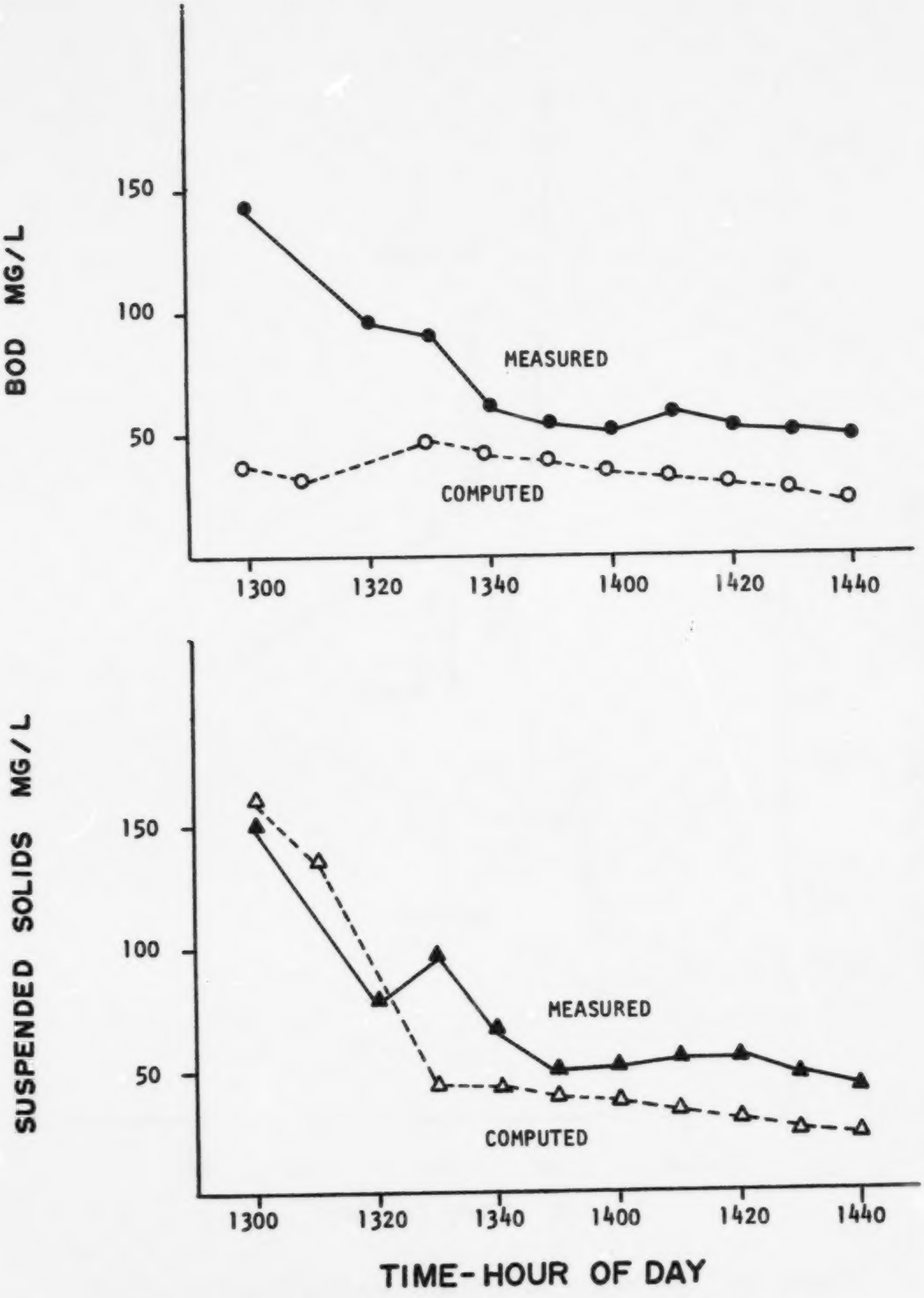


Figure 104. Run Number 30 effluent quality Site 1.

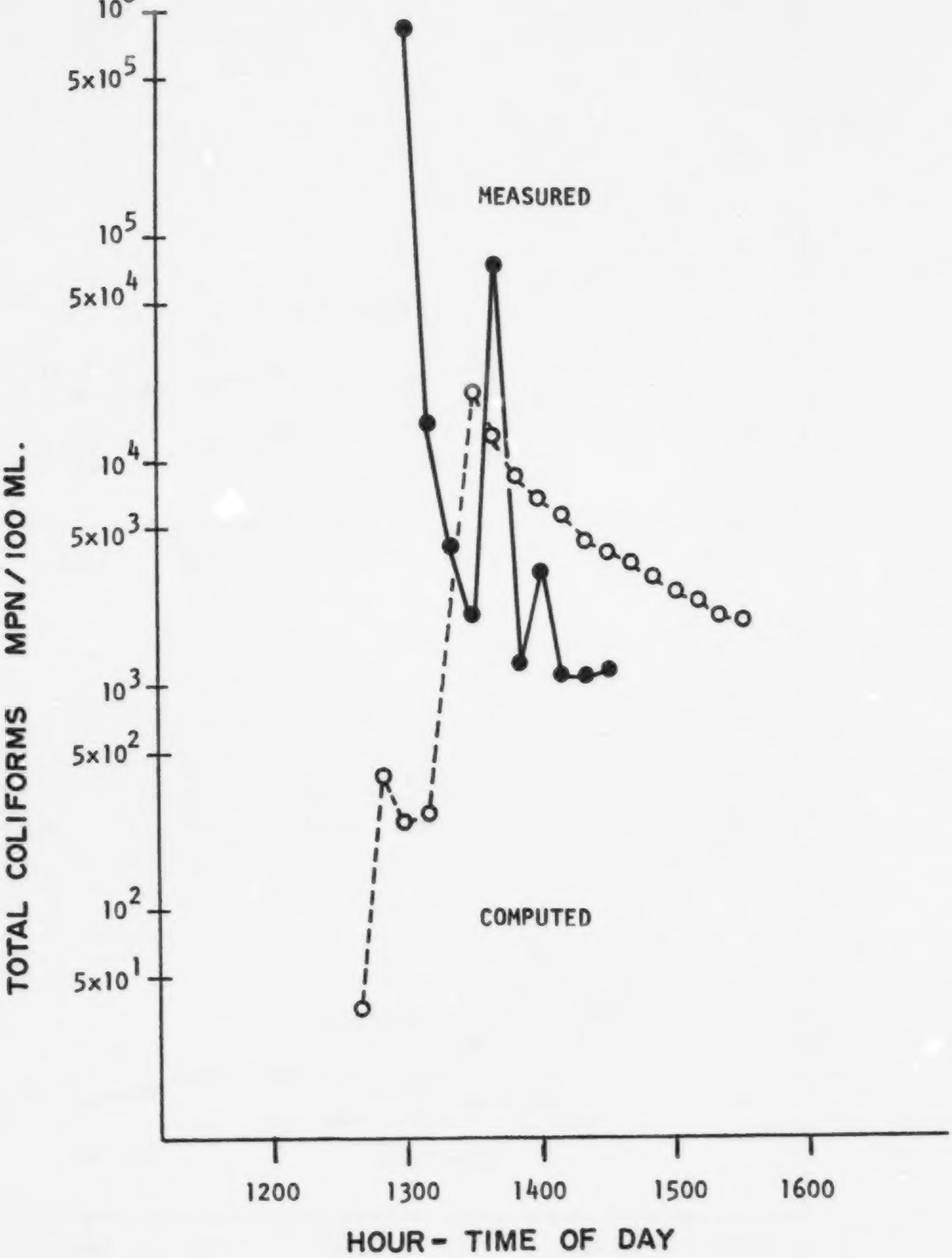


Figure 105. Run Number 30 effluent quality Site I.

listed in Table 104 and graphically represented in Figure 106. Because of the small amount of rain and because it was desired to use the Site II facilities at as high a flow as possible, the sluice gate was closed shortly after the start of the overflow. The computed values of the flow peak before the measured and at a slightly higher flow.

The ratio of the actual total flow arriving to the completed total flow is 1.13. The slightly longer duration of the actual flow results from the sluice gate being closed. The goodness of fit of the hydrograph is acceptable.

The arriving quality at Site II is determined by seven discrete samples taken at various intervals throughout the run. Table 105 lists the computed and actual concentrations for this run and Figures 107 and 108 present the graphical comparisons of these values. The BOD comparison in Figure 107 shows the "first flush" of the measured values while the computed concentrations remain relatively constant. The computed concentrations do not start until 1240 and no "first flush" was found. The suspended solids comparisons also show high initial measured values and a peak in the computed values. The goodness of fit of the suspended solids data is generally poor because of the computed peak at 1300 hours. The coliform concentrations in Figure 108 show that the computed values are lower throughout the run and show a large decrease at 1300 hours. Generally, the coliform comparisons are poor and the overall goodness of fit of the quality data is fair.

The effluent concentrations of BOD and suspended solids are listed in Table 106. Because of chlorination problems at Site II, no coliform data were obtained. Figure 109 presents the graphical comparison of the effluent quality. The BOD comparison shows the measured values to be generally constant at 20 mg/l while the computed values are higher with some variation. Because of the low effluent concentrations, the variations of less than 10 mg/l are insignificant. The suspended solids data shows very little correlation and even the trend of each is different. The computed influent suspended solids concentrations were well above 600 mg/l with the measured values at 300 mg/l. The same trend is found in the effluent concentrations with no difference in the closeness of fit. Because of the large differences in the arriving quality, little if any estimation of the Storage block effectiveness is possible. The goodness of fit of the effluent quality is poor.

Run Number 37

The next discretely sampled run was Number 37 which occurred on June 6, 1974. The average rainfall from the three raingauges of the area was 1.62 centimeters (0.64 inches) with a duration of 330 minutes. There was one dry day prior to this run and 16 days in which the cumulative rainfall was less than 2.54 centimeters (1 inch). Table E8, Appendix VI-E, lists the rainfall intensities used for this run. The computed and measured arriving flows for Site I are listed in Table 107 and graphically presented in

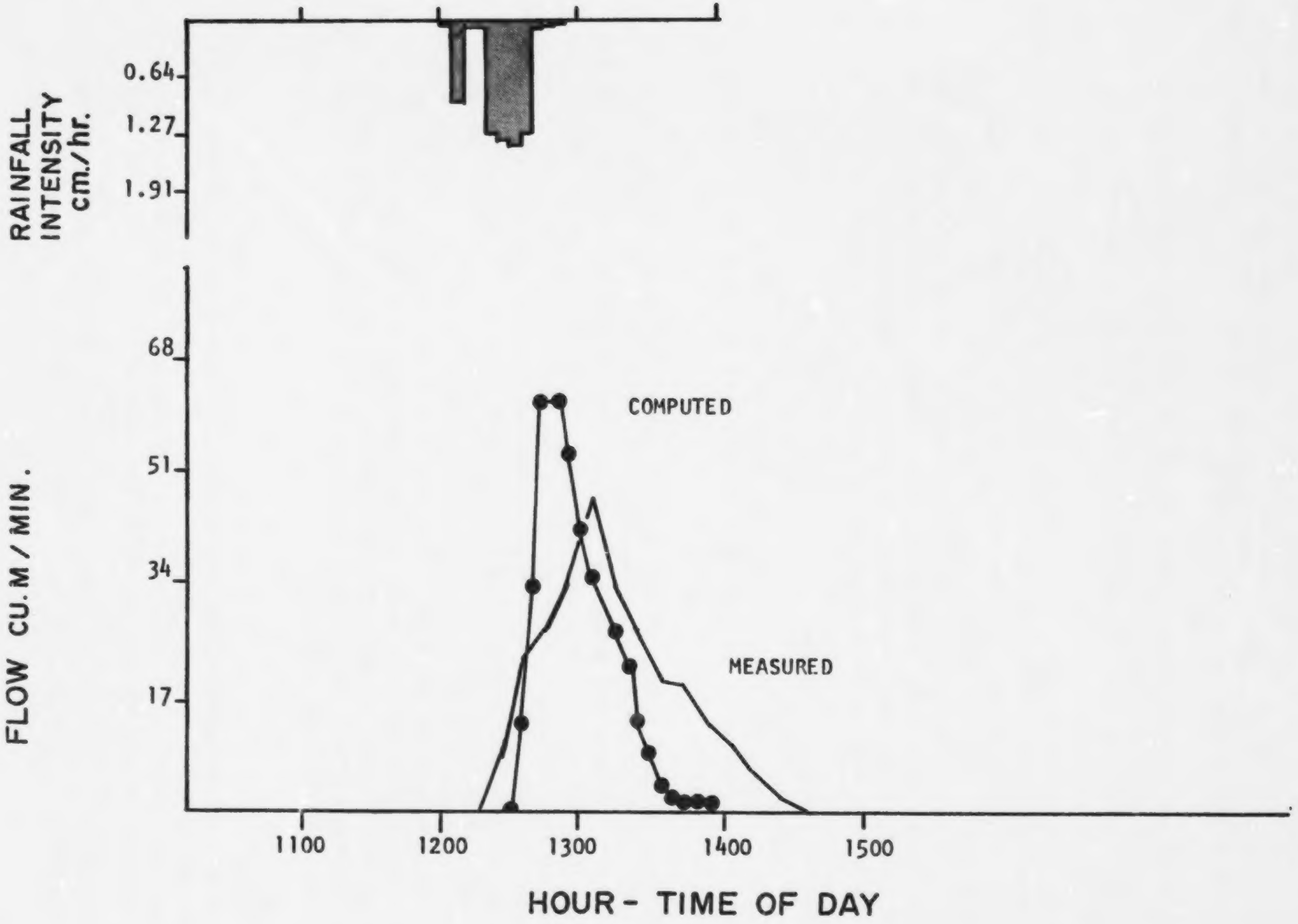


Figure 106. Run Number 30 arriving flow Site II.

TABLE 104. ARRIVING FLOW, SITE II

Run Number 30

Time hours	Arriving flow, cu m/min	cfs	Computed flow, cu m/min	cfs
1145	0.0	0.0		
1200	0.0	0.0		
	0.0	0.0		
1215				
	27.7	16.3		
1230	34.9	20.5	0.0	0.0
			0.9	0.5
	48.8	28.7	15.3	9.0
1245			34.5	20.3
	34.9	20.5	63.6	37.4
			63.8	37.5
1300	27.7	16.3	54.9	32.3
			43.9	25.8
	20.9	12.3	34.9	20.5
1315			28.1	16.5
	19.2	11.3	22.1	13.0
			14.5	8.5
1330	12.2	7.2	8.7	5.1
			4.6	2.7
	10.4	6.1	3.4	2.0
1345			3.2	1.9
	7.0	4.1	2.9	1.7
			2.8	1.6
1400	3.4	2.0	2.4	1.4
			2.4	1.4
	2.7	1.6	2.2	1.3
1415			2.0	1.2
	1.7	1.0	1.7	1.0
1430	1.7	1.0		

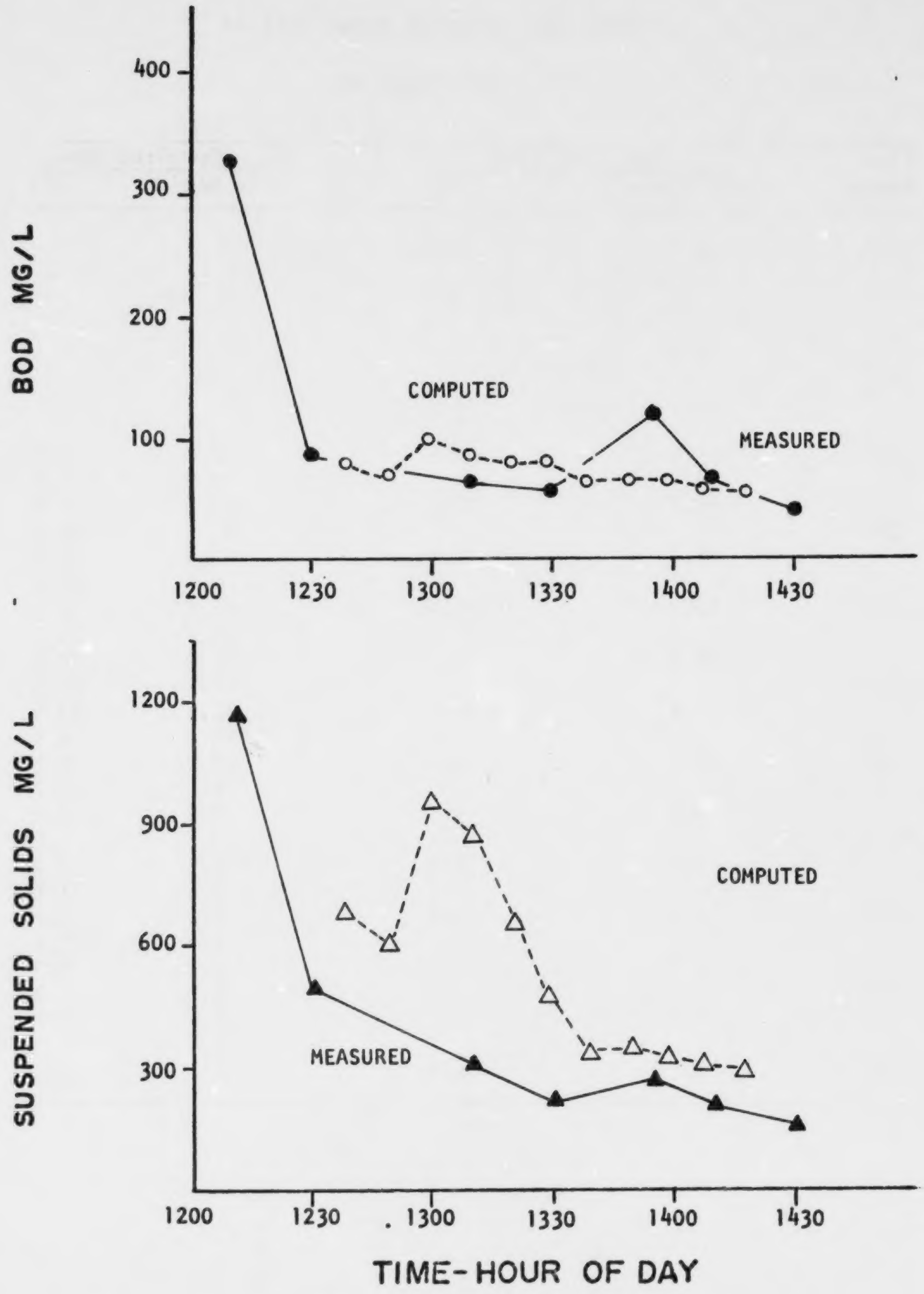


Figure 107. Run Number 30 arriving quality Site II.

TABLE 105. ARRIVING QUALITY, SITE 11

Run Number 30

Time hours	Actual BOD mg/l	Actual SS mg/l	Actual Coliforms No./100 ml	Computed BOD mg/l	Computed SS mg/l	Computed Coliforms MPN/100 ml
1200		1169				
	328		2.1×10^7			
1230	86	496	2.2×10^7			
				80	695	7.3×10^6
				70	608	1.7×10^6
				100	969	1.1×10^6
1300						
	65	310	2.1×10^7	89	896	1.8×10^6
				80	661	5.9×10^6
1330	57	220	2.7×10^7	80	496	7.1×10^6
				65	352	5.3×10^6
				68	351	4.9×10^6
1400	119	265	1.3×10^7	64	338	4.3×10^6
	66	198	2.8×10^7	61	325	4.0×10^6
				58	310	3.8×10^6
1430	40	156	1.8×10^7			

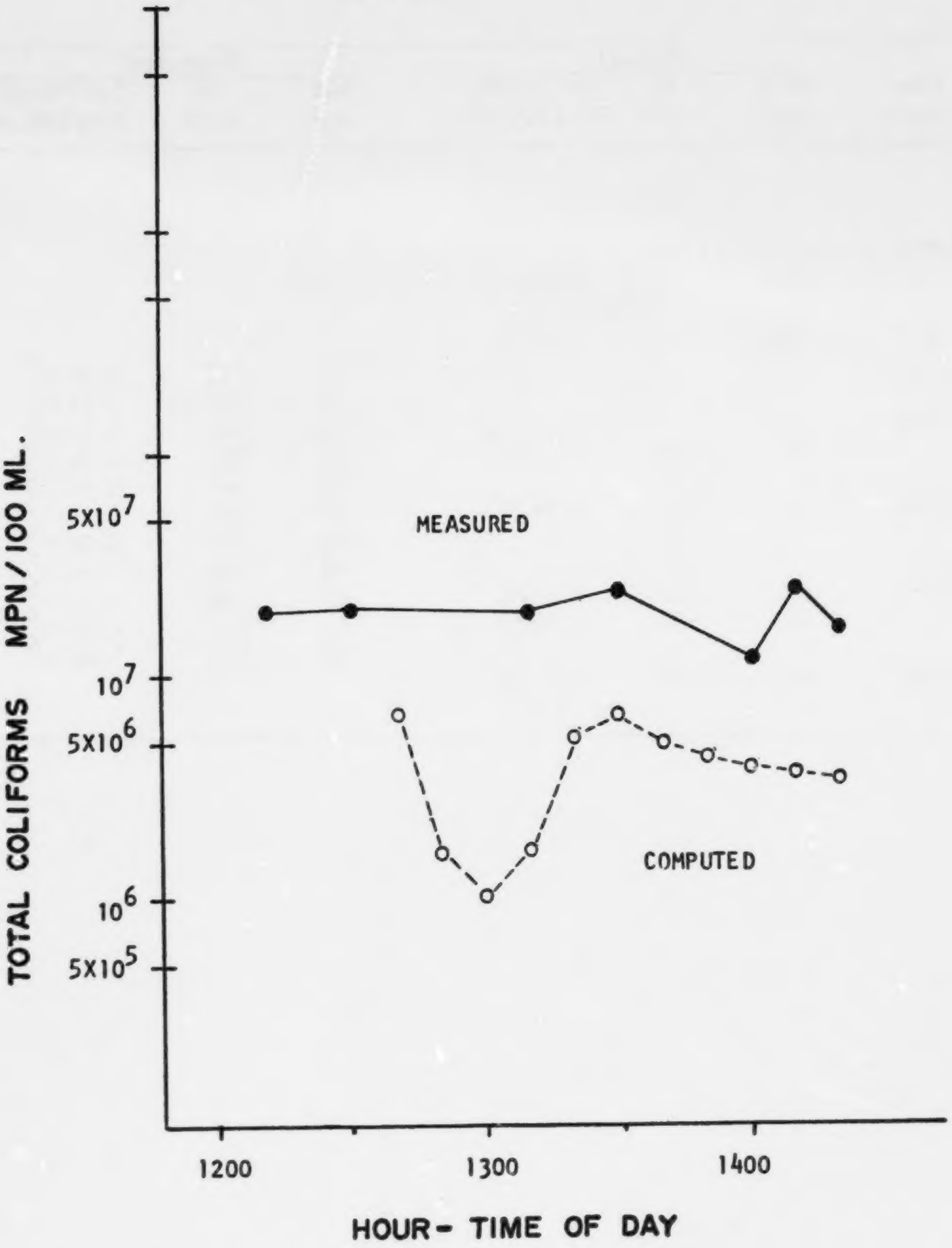


Figure 108. Run Number 30 arriving quality Site II.

TABLE 106. EFFLUENT QUALITY, SITE II

Run Number 30

Time hours	Actual BOD mg/l	Actual SS mg/l	Actual Coliforms[a] No./100 ml	Computed BOD mg/l	Computed SS mg/l	Computed Coliforms[a] MPN/100 ml
1200						
1215	19	339				
1230	20	102				
1245				28	10	
				35	18	
	17	97		30	28	
1300				39	87	
				42	133	
				37	163	
1315	16	101		37	173	
				33	170	
				33	171	
1330	16	57		31	170	
				32	170	
				29	146	
1345	16	62		26	138	
				27	141	
				30	141	
1400				27	143	
				26	147	
	18	66		24	151	
1415				24	150	
				22	91	
1430						
1435	23	53				

[a] No Coliform Data, Chlorination at Site II Inoperative.

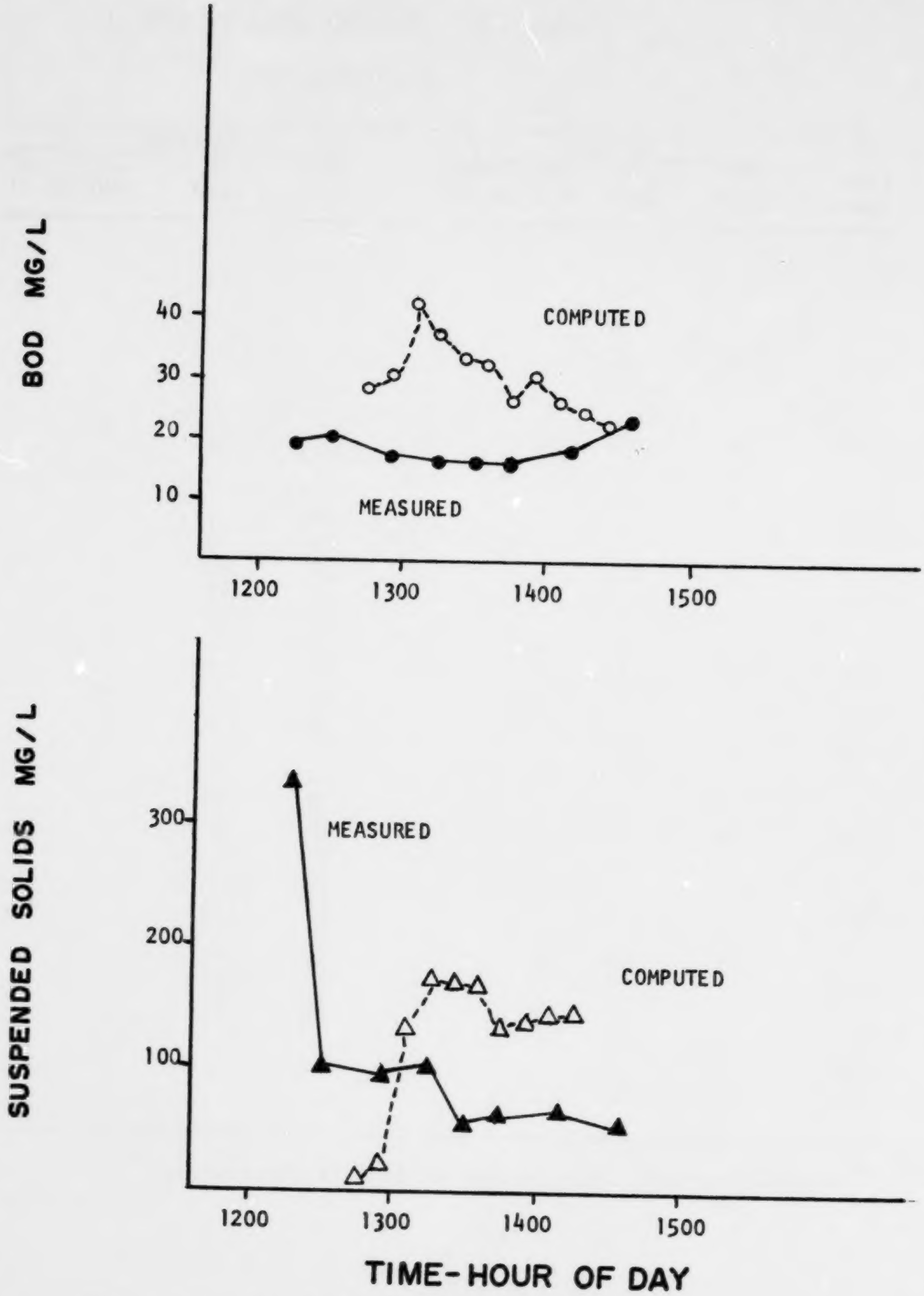


Figure 109. Run Number 30 effluent quality Site II.

TABLE 107. ARRIVING FLOW, SITE I

Run Number 37

Time hours	Arriving flow, cu m/min min.	Arriving flow, cu m/min max.	Arriving flow, cfs min.	Arriving flow, cfs max.	Computed flow, cu m/min.	Computed flow, cfs
1700	0.0	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0
	10.4	10.4	6.1	6.1	0.0	0.0
1730	43.2	46.2	25.4	27.2	4.1	2.4
	42.8	47.4	25.2	27.9	17.0	10.0
	40.8	49.1	24.0	28.9	11.2	6.6
1800	37.1	45.4	21.8	26.7	14.3	8.4
	37.1	45.4	21.8	26.7	18.4	10.8
	37.1	45.4	21.8	26.7	38.1	22.4
1830	37.1	45.4	21.8	26.7	51.3	30.2
	37.1	45.4	21.8	26.7	56.1	33.0
	37.1	45.4	21.8	26.7	56.6	33.3
1900	24.1	27.2	14.2	16.0	53.9	31.7
	20.7	25.5	12.2	15.0	50.5	29.7
	19.7	23.8	11.6	14.0	47.3	27.6
1930	17.0	22.1	10.0	13.0	44.5	26.2
	16.0	22.1	9.4	13.0	42.8	25.2
	13.6	20.1	8.0	11.8	42.3	24.9
2000	12.1	16.7	7.1	9.8	43.2	25.4
	10.7	15.5	6.3	9.1	43.7	25.7
	9.2	13.9	5.4	8.2	44.0	25.9
2030	8.3	12.1	4.9	7.1	43.0	25.3
	8.0	11.9	4.7	7.0	41.7	24.5
	7.3	11.1	4.3	6.5	37.1	21.8
2100	6.8	9.9	4.0	5.8	31.3	18.4
	6.5	9.4	3.8	5.5	25.7	15.1
	5.8	8.8	3.4	5.2	20.9	12.3
2130	5.3	8.3	3.1	4.9	17.3	10.2
	7.1	11.4	4.2	6.7	14.6	8.6
	9.2	14.3	5.4	8.4	12.8	7.5
2200	11.4	15.1	6.7	8.9	11.7	6.9
	11.9	14.3	7.0	8.4	11.4	6.7
	12.4	16.8	7.3	9.9	11.7	6.9
2230	12.8	17.3	7.5	10.2	12.4	7.3
	12.4	16.7	7.3	9.8	12.2	7.2
	12.1	15.6	7.1	9.2	10.9	6.4
2300	12.1	15.1	7.1	8.9	9.2	5.4
	10.5	13.9	6.2	8.2	7.3	4.3
	9.2	12.6	5.4	7.4	5.6	3.3
2330	7.8	11.4	4.6	6.7	3.9	2.3
	8.0	11.6	4.7	6.8	2.6	1.5
	8.2	11.7	4.8	6.9	1.7	1.0
2400	8.3	12.1	4.9	7.1	0.0	0.0

Figure 110. This figure shows that the computed flow again lags behind the measured by about 30 minutes. The computed flows also remain higher after the initial peak is reached, possibly because of the surcharging of some elements within the transport network. The ratio of the total measured arriving flow to the total computed flow is 1.3. The goodness of fit of the two hydrographs is fair.

The computed and arriving quality is listed in Table 108 and graphically represented in Figures 111 and 112. As Figure 111 indicates, the measured BOD and suspended solids concentrations increase as the run progresses while the computed values decrease. Since there was only one dry day prior to this run, the computed BOD values are very low initially and decrease rapidly to near zero. The suspended solids data are initially high and again decrease to near zero. There is little, if any correlation between the computed and actual values.

The coliform data presented in Figure 112 shows the computed coliform concentrations to be low initially and then stabilize at higher levels. The measured values show just the opposite trend with much variation between values. The overall goodness of fit of the arriving quality graphs is poor because of the differences in trend prediction.

The effluent values for Site I during this run are listed in Table 109 and graphed in Figures 113 and 114. Because of the large differences in the actual and computed arriving BOD, suspended solids and coliform concentrations, these differences are carried through the Storage block and make any analysis of the output difficult. The coliform data in Figure 115 shows little correlation or similarity.

During the operation of the Site II treatment unit, the recording pen of the plant flow chart did not operate for the first hour of operation. Since the sites start-up automatically, the pen was not fixed until operating personnel arrived after one hour of the overflow had passed, but samples of the incoming flow were automatically obtained. Table 110 lists the computed and actual flows for this run and Figure 115 presents the graphical comparison. No evaluation of the arriving flow is possible because of the mechanical problems but the quality data is useful.

Table 111 lists the computed and actual concentration of the arriving flow at Site II and Figures 116 and 117 present this data in graphical form. Note that no computed values are presented after 1900 hours since the computed flow at this time was approaching zero. The BOD values are relatively accurate in predicting the arriving quality while the suspended solids concentrations are variable and show little correlation. The coliform data presented in Figure 117 shows the computed values to be one order of magnitude lower than the measured values. This difference is not due to the methods of analysis between the computed and the actual coliform numbers inasmuch as the two methods (membrane filter and MPN) were not found to differ appreciably during this run. The two curves are very similar in trend and show good correlation. The overall goodness of

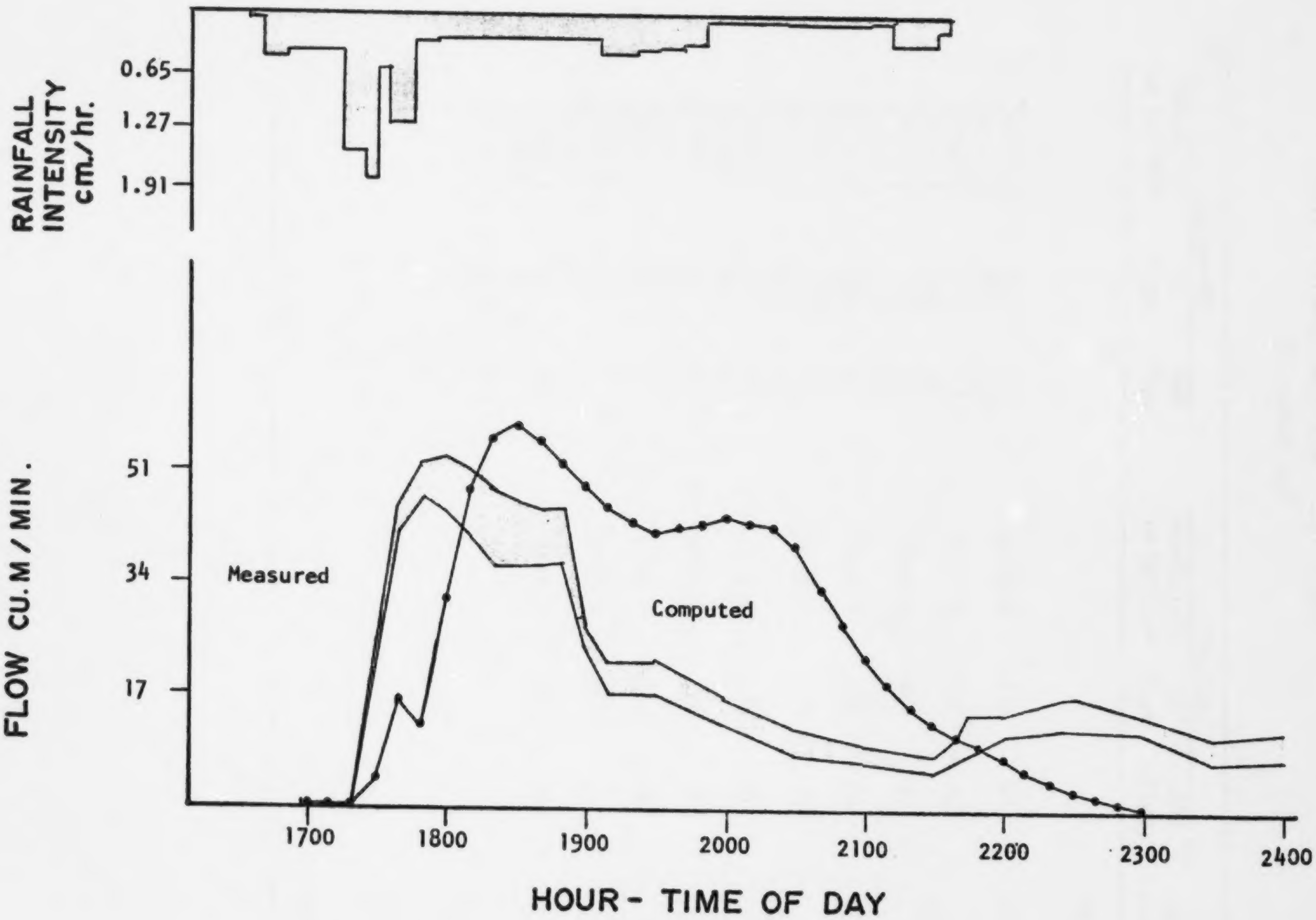


Figure 110. Run Number 37 arriving flow Site 1.

TABLE 108. ARRIVING QUALITY, SITE I

Run Number 37

	Actual			Computed		
Time hours	BOD mg/l	SS mg/l	Coliforms No./100 ml	BOD mg/l	SS mg/l	Coliforms MPN/100 ml
1700						
1730	52	213	2.4×10^7	93	769	7.7×10^4
				69	565	2.5×10^5
	136	405	2.4×10^7	61	713	4.0×10^5
1800				43	568	7.4×10^5
	179	474	2.4×10^8	68	387	5.6×10^6
				75	402	4.5×10^6
1830	64	365	3.9×10^6	63	388	2.9×10^6
				49	330	2.1×10^6
	80	314	4.3×10^6	37	260	1.7×10^6
1900				27	196	1.5×10^6
	71	243	9.3×10^6	20	143	1.4×10^6
				16	105	1.3×10^6
1930	131	347	2.4×10^7	13	78	1.2×10^6
				11	61	1.1×10^6
	135	463	4.3×10^6	9	50	1.1×10^6
2000				8	39	1.1×10^6
	126	400	9.0×10^6	7	32	1.1×10^6
				6	25	1.1×10^6
2030	195	664	2.4×10^7	6	19	1.2×10^6
				6	16	1.4×10^6
				6	15	1.5×10^6
2100				6	13	1.8×10^6
2130						
2200						

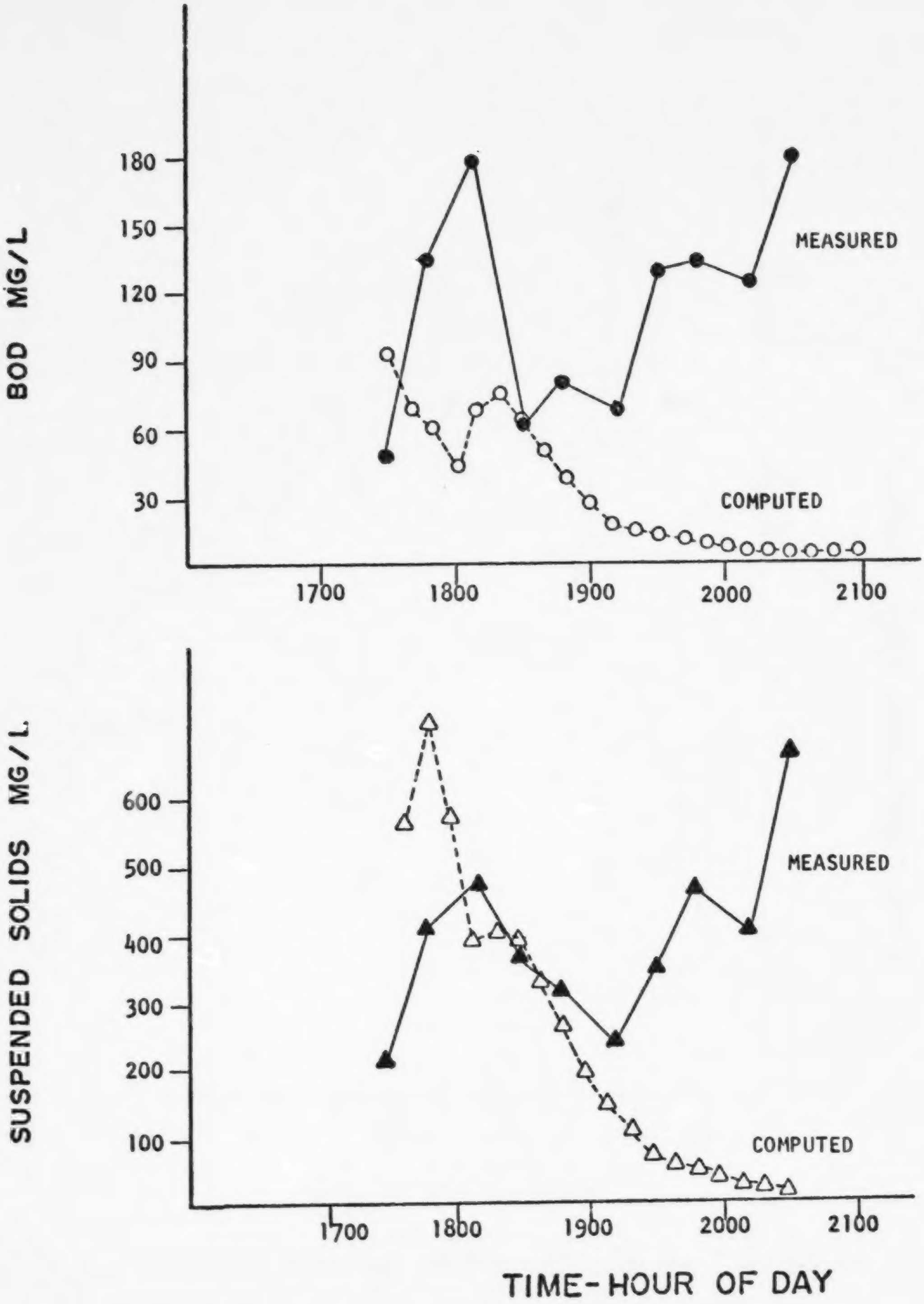


Figure III. Run Number 37 arriving quality Site I.

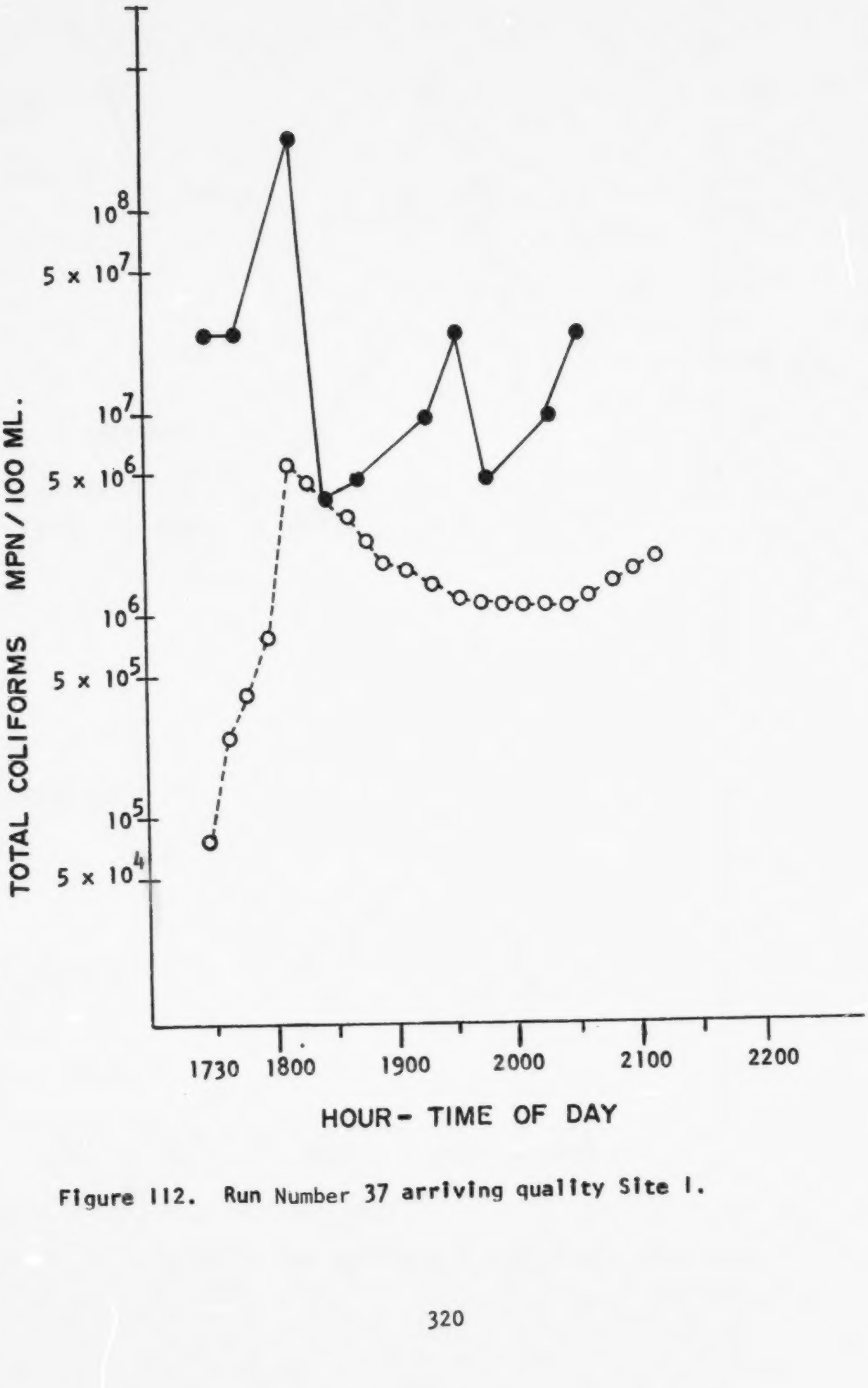


Figure 112. Run Number 37 arriving quality Site 1.

TABLE 109. EFFLUENT QUALITY, SITE I

Run Number 37

Time hours	Actual BOD mg/l	Actual SS mg/l	Actual Coliforms No./100 ml	Computed BOD mg/l	Computed SS mg/l	Computed Coliforms MPN/100 ml
1800	42	210	8.6×10^3	38	142	7.6×10^1
				28	104	2.5×10^2
	19	76	4.8×10^5	25	132	4.0×10^2
1830				18	105	7.3×10^2
	11	41	4.8×10^3	28	108	5.5×10^3
				39	187	8.7×10^5
1900	11	32	4.8×10^4	38	215	9.6×10^5
				30	190	7.5×10^5
	25	75	1.9×10^4	23	150	5.8×10^5
1930				16	108	4.5×10^5
	30	103	4.2×10^4	11	75	3.4×10^5
				8	52	2.5×10^5
2000	31	72	2.2×10^5	6	36	1.8×10^5
				5	27	1.5×10^5
	33	73	4.8×10^5	5	22	1.5×10^5
2030				4	17	1.7×10^5
	35	102	4.8×10^5	4	14	1.9×10^5
				3	10	1.8×10^5
2100	70	223	4.8×10^5	3	7	1.7×10^5

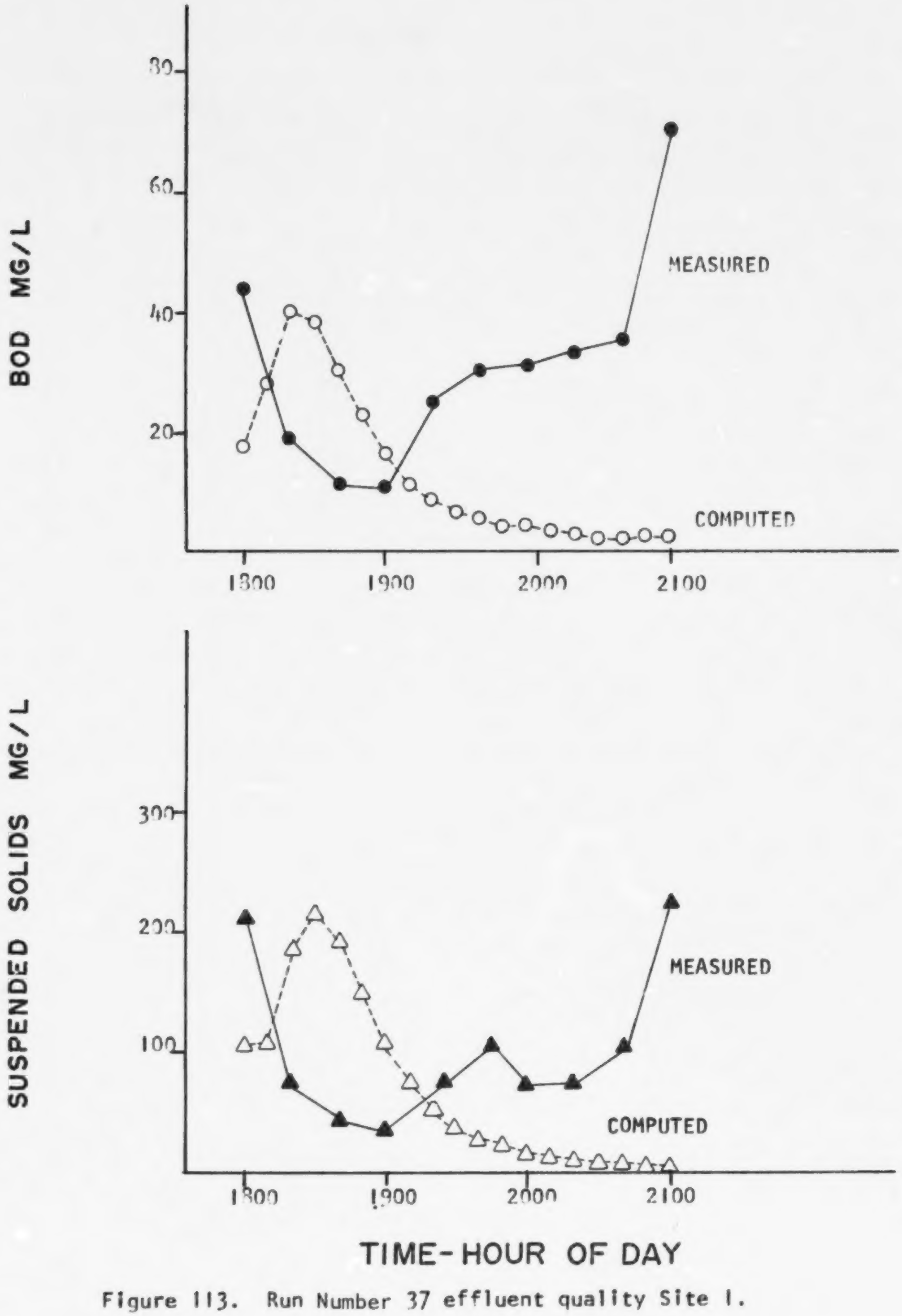


Figure 113. Run Number 37 effluent quality Site I.

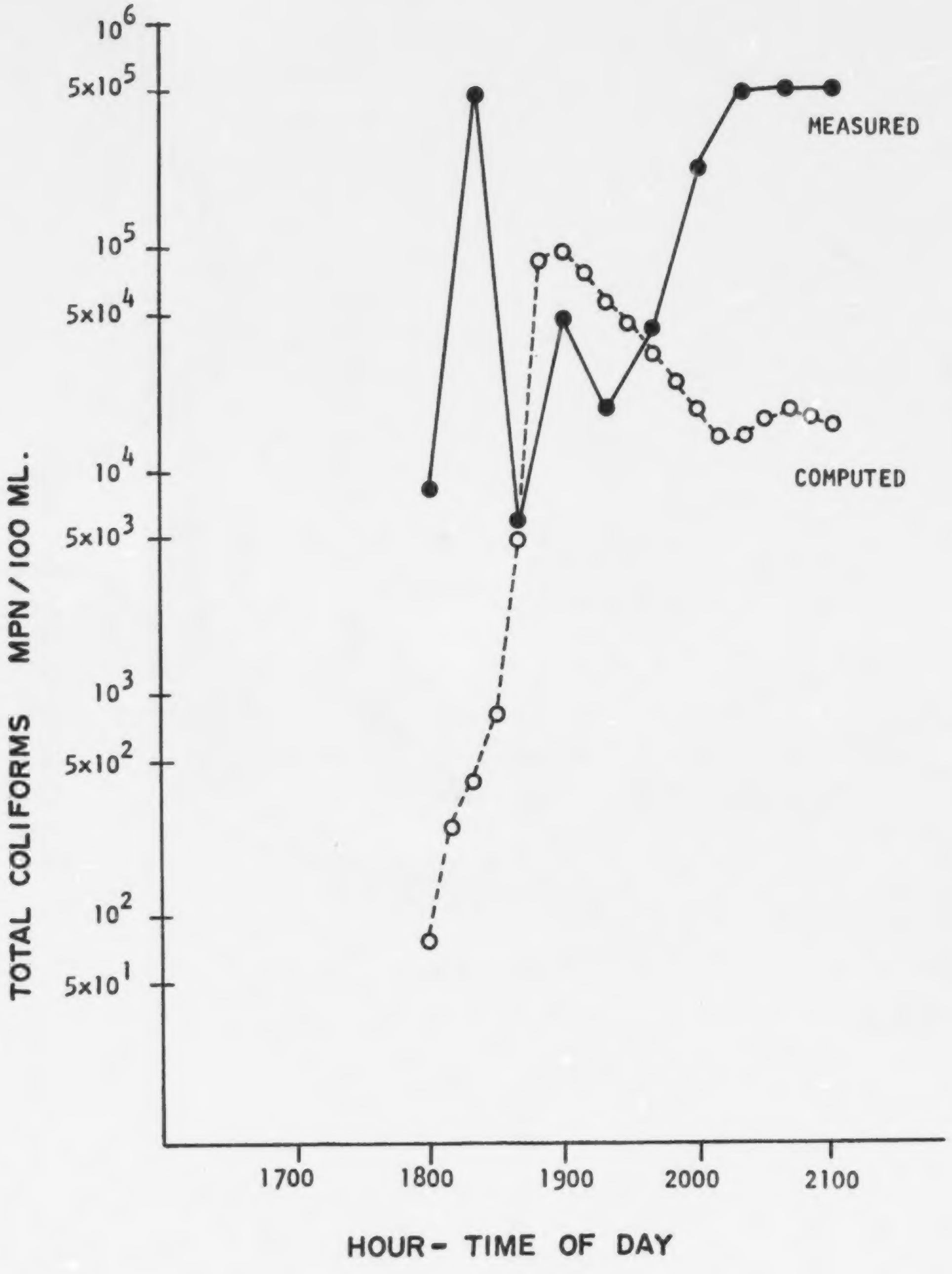


Figure 114. Run Number 37 effluent quality Site 1.

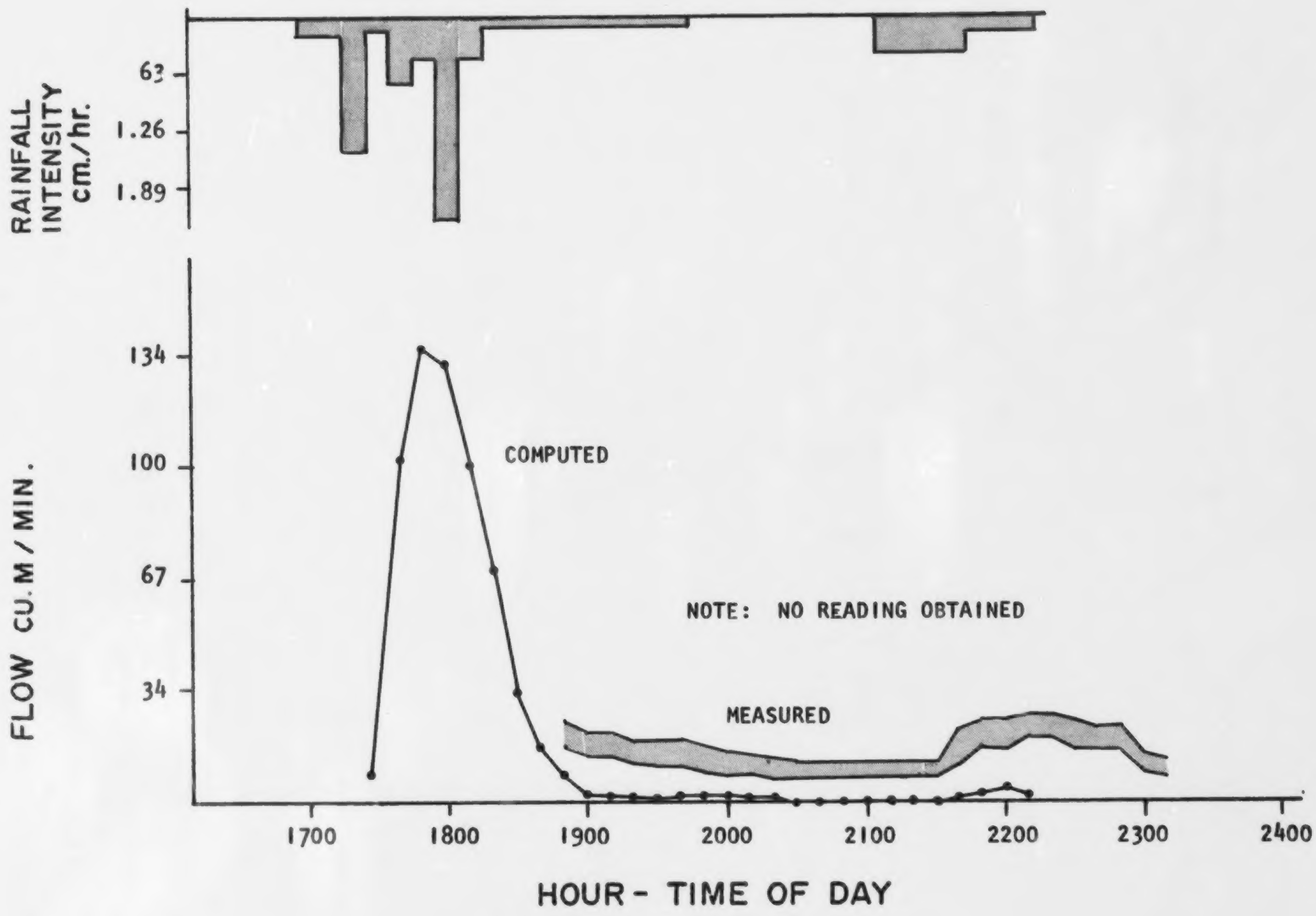


Figure 115. Run Number 37 arriving flow Site 11.

TABLE 110. ARRIVING FLOW, SITE II

Run Number 37

Time	Arriving flow,				Computed flow,	
	cu m/min		cfs		cu m/min.	cfs
hours	min.	max.	min.	max.		
1700						
1730					8.3	4.9
					103.7	61.0
					137.5	80.9
1800					132.4	77.9
					102.0	60.0
					70.4	41.4
1830					33.2	19.5
					16.0	9.4
					7.3	4.3
1900	13.9	20.9	8.2	12.3	2.0	1.2
	13.9	20.9	8.2	12.3	1.8	1.1
	12.2	19.2	7.2	11.3	1.3	0.8
1930	11.4	19.2	6.7	11.3	1.0	0.6
	11.4	19.2	6.7	11.3	1.3	0.8
	10.3	17.3	6.1	10.2	1.7	1.0
2000	8.7	15.6	5.1	9.2	1.8	1.1
	8.7	14.8	5.1	8.7	1.0	0.6
	6.9	13.9	4.1	8.2	0.6	0.4
2030	6.9	12.2	4.1	7.2	0	0
	6.9	12.2	4.1	7.2	0	0
	6.9	12.2	4.1	7.2	0	0
2100	6.9	12.2	4.1	7.2	0	0
	6.9	12.2	4.1	7.2	0	0
	6.9	12.2	4.1	7.2	0	0
2130	6.9	12.2	4.1	7.2	0	0
	10.4	22.6	6.1	13.3	1.2	0.7
	15.6	24.3	9.2	14.3	3.7	2.2
2200	15.6	24.3	9.2	14.3	3.7	2.2
	19.2	26.0	11.3	15.3	2.2	1.3
	19.2	26.0	11.3	15.3	0	0
2230	15.6	24.3	9.2	14.3	0	0
	15.6	22.6	9.2	13.3	0	0
	15.4	22.4	9.1	13.2	0	0
2300	8.6	14.0	5.1	8.2	0	0
	7.0	12.2	4.1	7.2	0	0

TABLE 111. ARRIVING QUALITY, SITE II

Run Number 37

	Actual			Computed		
Time hours	**BOD mg/l**	**SS mg/l**	**Coliforms No./100 ml**	**BOD mg/l**	**SS mg/l**	**Coliforms MPN/100 ml**
1700						
1730	56	494	4.6×10^7	53	79	2.1×10^6
				67	414	1.0×10^6
	116	780	4.3×10^6	50	460	5.3×10^5
1800				38	457	4.3×10^5
	45	655	9.3×10^6	30	382	7.9×10^5
				28	376	1.0×10^6
1830	33	355	9.3×10^6	28	436	1.3×10^6
				25	466	1.5×10^6
				20	254	1.7×10^6
1900				20	650	1.4×10^6
	51	428	1.5×10^7			
1930	42	396	9.3×10^6			
	29	361	4.3×10^6			
2000	27	345	2.4×10^7			
	29	331	1.5×10^7			
2030	26	314				

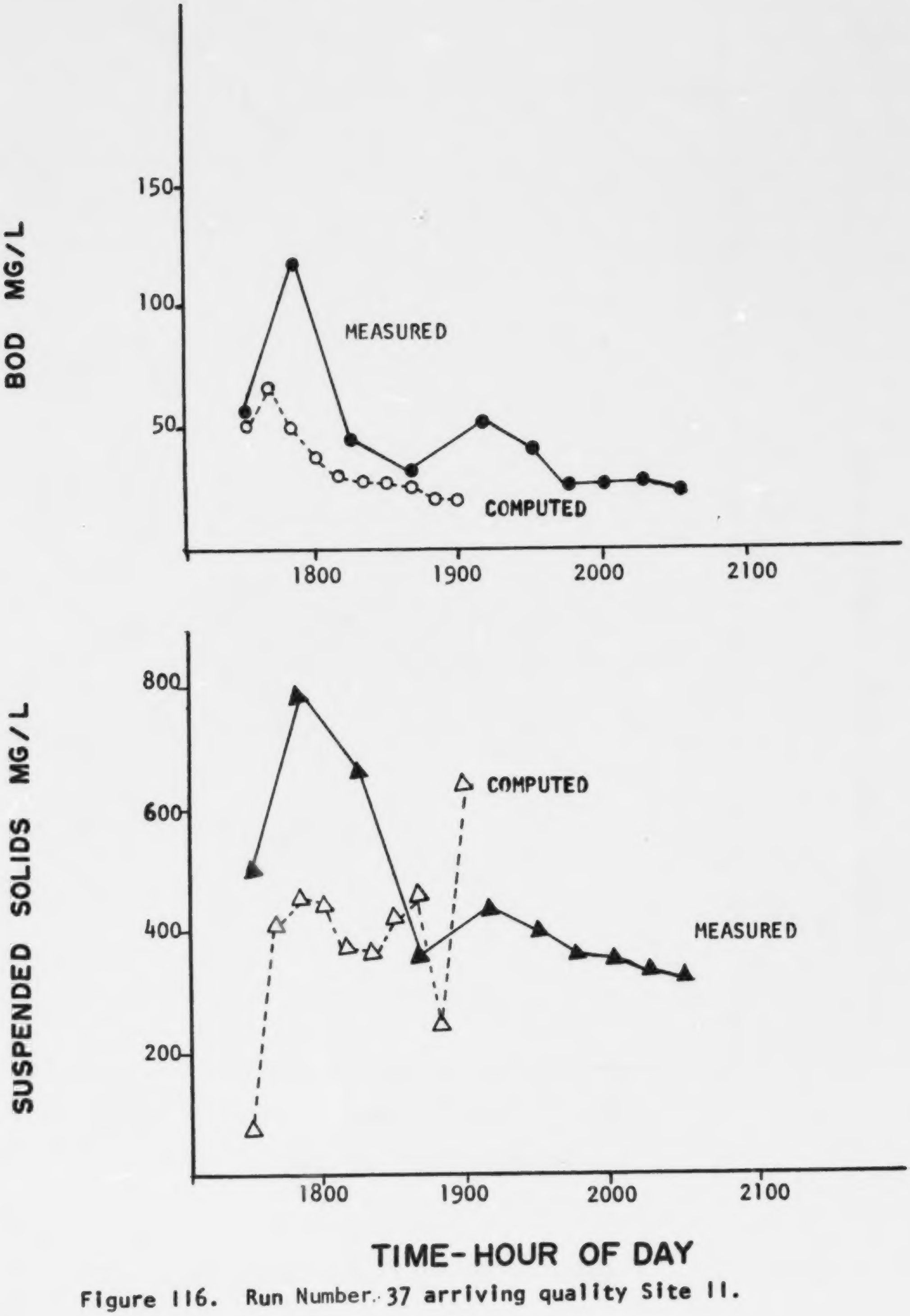


Figure 116. Run Number 37 arriving quality Site 11.

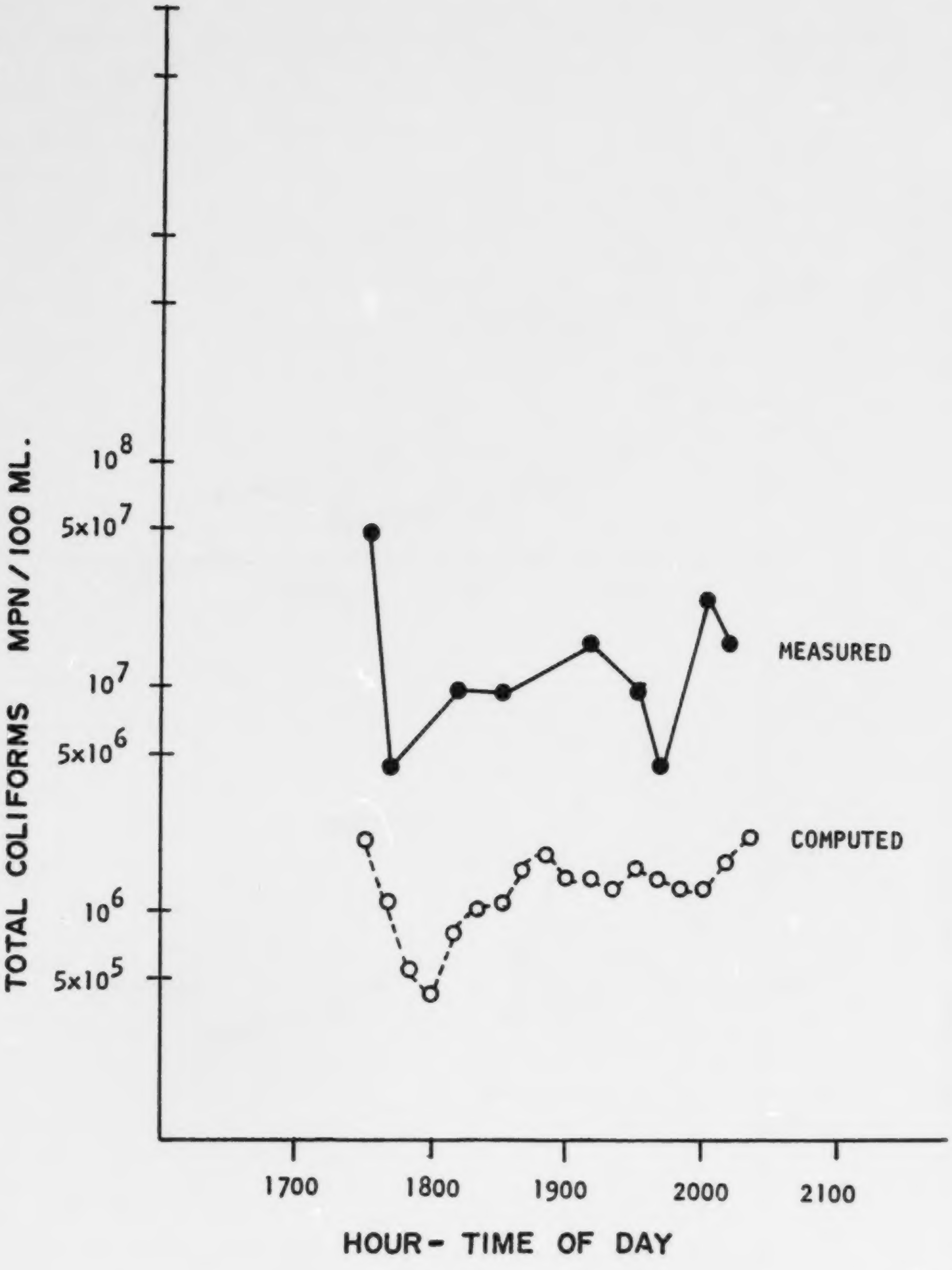


Figure 117. Run Number 37 arriving quality Site II.

fit of the quality curves is therefore fair.

The actual and computed values of the effluent from the Site II treatment unit is listed in Table 112 and graphically represented in Figure 118. These comparisons show very little similarity because of the low arriving concentrations. The resulting goodness of fit is, therefore, poor.

Run Number 45

The seventh and final series sampled run was Number 45 which occurred on August 16, 1974. The average rainfall for the entire drainage area was 1.75 centimeters (0.69 inch) over 350 minutes of noncontinuous rainfall. There are three small rainfall events within this 350 minute duration. This pattern of rainfall provided an opportunity to simulate the storms that are typical for the late Summer and Fall seasons in the Wisconsin area. The rainfall intensities at 5-min intervals for this run are listed in Table E9, Appendix VI-E. The computed and measured arriving flows to Site I are listed in Table 113 and plotted in Figure 119. The computed and measured flows are very close throughout the first four hours of the run. After this initial low flow, the computed flow peaks well above the measured at generally the same point in time. The ratio of the total measured flow arriving to the total computed flow is 0.53. The closeness of fit of the two curves is generally good. Although the magnitudes of the final peaks are different, the two records are similar in trend.

The quality of the arriving flow was determined by eight discrete samples taken during the final portion of the overflow where a majority of the total flow for this run occurred. Table 114 lists the actual and computed concentrations for this run and Figures 120 and 121 present the graphical comparison. The arriving BOD of Figure 120 show that the computed values decrease rapidly from the start of the overflow. The measured values are generally higher during the later stages of the overflow. The suspended solids concentrations show the same pattern but the computed values show a slight peak at the final stages of the run. These comparisons tend to indicate that the SWMM predicts most of the arriving BOD and suspended solids during the first hours of the overflow. Because no actual samples were taken during the first hours, it is hard to predict what the total actual pollutographs would be. Thus, the goodness of fit of the two pollutographs is poor. The coliform data plotted in Figure 121 show the measured and computed values to be relatively constant throughout the overflow. The measured values are greater in number than the computed. The closeness of fit is therefore, good and overall goodness of fit of the quality data is fair.

The effluent from the treatment unit was also discretely sampled during the final portion of this run. Table 115 lists the measured and computed effluent concentrations and Figures 122 and 123 present these results in graphical form. Because of the large differences in the computed and actual influent concentrations, little if any comparison of the effluent values can be made. The actual coliform concentrations are extremely low

TABLE 112. EFFLUENT QUALITY, SITE II

Run Number 37

Time hours	Actual			Computed		
	BOD mg/l	SS mg/l	Coliforms No./100 ml	BOD mg/l	SS mg/l	Coliforms MPN/100 ml
			*			
1740				28	172	$2.08X10^3$
				25	233	$1.00X10^3$
1800				18	195	$8.29X10^4$
				12	120	$5.30X10^4$
				12	83	$7.70X10^2$
1830				12	60	$1.00X10^3$
				10	48	$1.20X10^3$
				9	34	$1.48x10^3$
1900	24	89		6	22	$1.63x10^3$
				6	18	$1.32x10^3$
	16	199		6	18	$1.29x10^3$
1930				7	20	$1.30x10^3$
	17	107		6	15	$1.40x10^3$
	21	136		5	13	$1.27x10^3$
2000	22	235		5	10	$1.22x10^3$
				5	14	$1.24x10^3$
	23	228		7	21	$1.48x10^3$
2030						
	18	107				
	20	92				
2100						
	14	101				

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* No coliform analysis run due to chlorination problems.

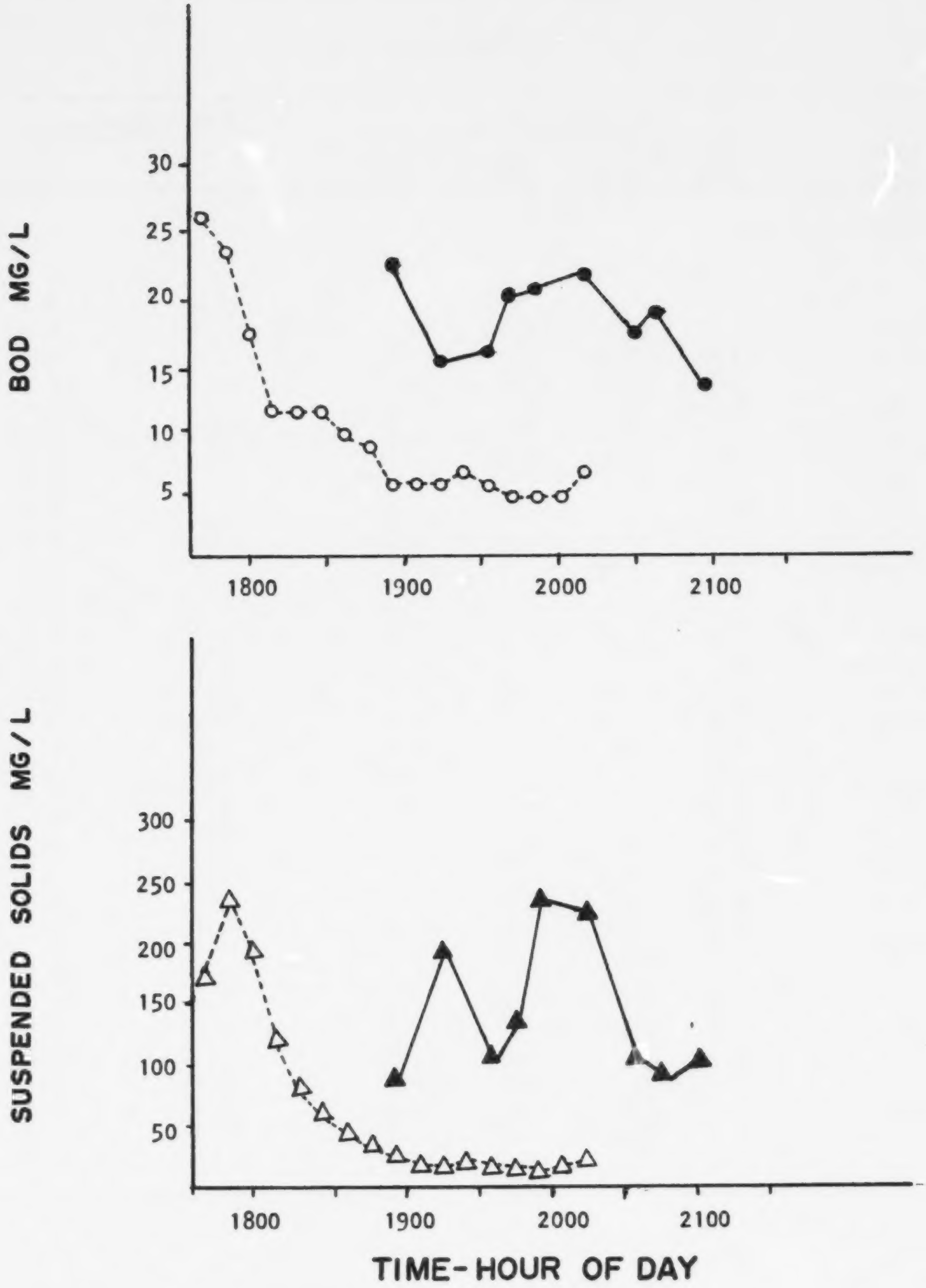

Figure 118. Run Number 37 effluent quality Site II.

TABLE 113. ARRIVING FLOW, SITE I

Run Number 45

Time hours	Arriving flow, cu m/min min.	Arriving flow, cu m/min max.	Arriving flow, cfs min.	Arriving flow, cfs max.	Computed flow, cu m/min.	Computed flow, cfs
1230						
					3.6	2.1
1300	14.3	19.7	8.4	11.6	3.4	2.0
	13.6	18.8	8.0	11.1	1.5	0.9
	9.8	12.9	5.8	7.6	0.0	0.0
1330	4.6	8.3	2.7	4.9	5.1	3.0
	3.7	8.3	2.2	4.9	9.3	5.5
	3.7	4.6	2.2	2.7	10.3	6.1
1400	3.7	4.6	2.2	2.7	9.3	5.5
	3.0	3.7	1.8	2.2	7.8	4.6
	3.0	3.7	1.8	2.2	6.0	3.5
1430	3.0	3.7	1.8	2.2	4.4	2.6
	3.4	4.0	2.0	2.4	3.5	2.1
	3.7	4.6	2.2	2.7	2.9	1.7
1500	4.6	5.2	2.7	3.1	3.2	1.9
	4.9	5.6	2.9	3.3	4.0	2.4
	5.2	6.1	3.1	3.6	5.1	3.0
1530	4.6	5.2	2.7	3.1	6.4	3.8
	3.0	3.7	1.8	2.2	6.8	4.0
	2.2	3.0	1.3	1.8	6.3	3.7
1600	4.6	5.2	2.7	3.1	5.1	3.0
	6.1	6.8	3.6	4.0	3.9	2.3
	8.3	9.0	4.9	5.3	2.5	1.5
1630	6.8	7.5	4.0	4.4	1.5	0.9
	5.2	6.1	3.1	3.6	2.2	1.3
	6.1	6.8	3.6	4.0	6.6	3.9
1700	7.5	8.3	4.4	4.9	10.7	6.3
	6.8	6.8	4.0	4.0	18.0	10.6
	6.8	11.4	4.0	6.7	28.9	17.0
1730	11.4	13.6	6.7	8.0	40.6	23.9
	15.1	16.6	8.9	9.8	50.6	29.8
	26.5	30.9	15.6	18.2	58.1	34.2
1800	26.5	30.9	15.6	18.2	63.6	37.4
	28.7	33.3	16.9	19.6	67.8	39.9
	28.7	33.3	16.9	19.6	70.4	41.4

TABLE 113. (continued). ARRIVING FLOW, SITE I

Run Number 45

Time	Arriving flow,				Computed flow,	
	cu m/min		cfs		cu m/min.	cfs
hours	min.	max.	min.	max.		
1830	28.7	33.3	16.9	19.6	67.5	39.7
	18.8	24.1	11.1	14.2	64.7	38.1
	14.2	15.1	8.4	8.9	62.7	36.5
1900	14.2	15.1	8.4	8.9	54.2	31.9
	6.1	9.0	3.6	5.3	44.9	26.4
	6.1	9.0	3.6	5.3	36.2	21.3
1930	6.1	9.0	3.6	5.3	29.2	17.2
	3.0	6.8	1.8	4.0	23.4	13.8
	3.0	6.8	1.8	4.0	18.9	11.1
2000	3.0	6.5	1.8	3.8	15.3	9.0
	2.2	5.2	1.3	3.1	12.2	7.2
					9.7	5.7
2030					7.5	4.4
					5.8	3.4
					4.0	2.4
2100					2.9	1.7
					1.7	1.0
					0.7	0.4
2130						

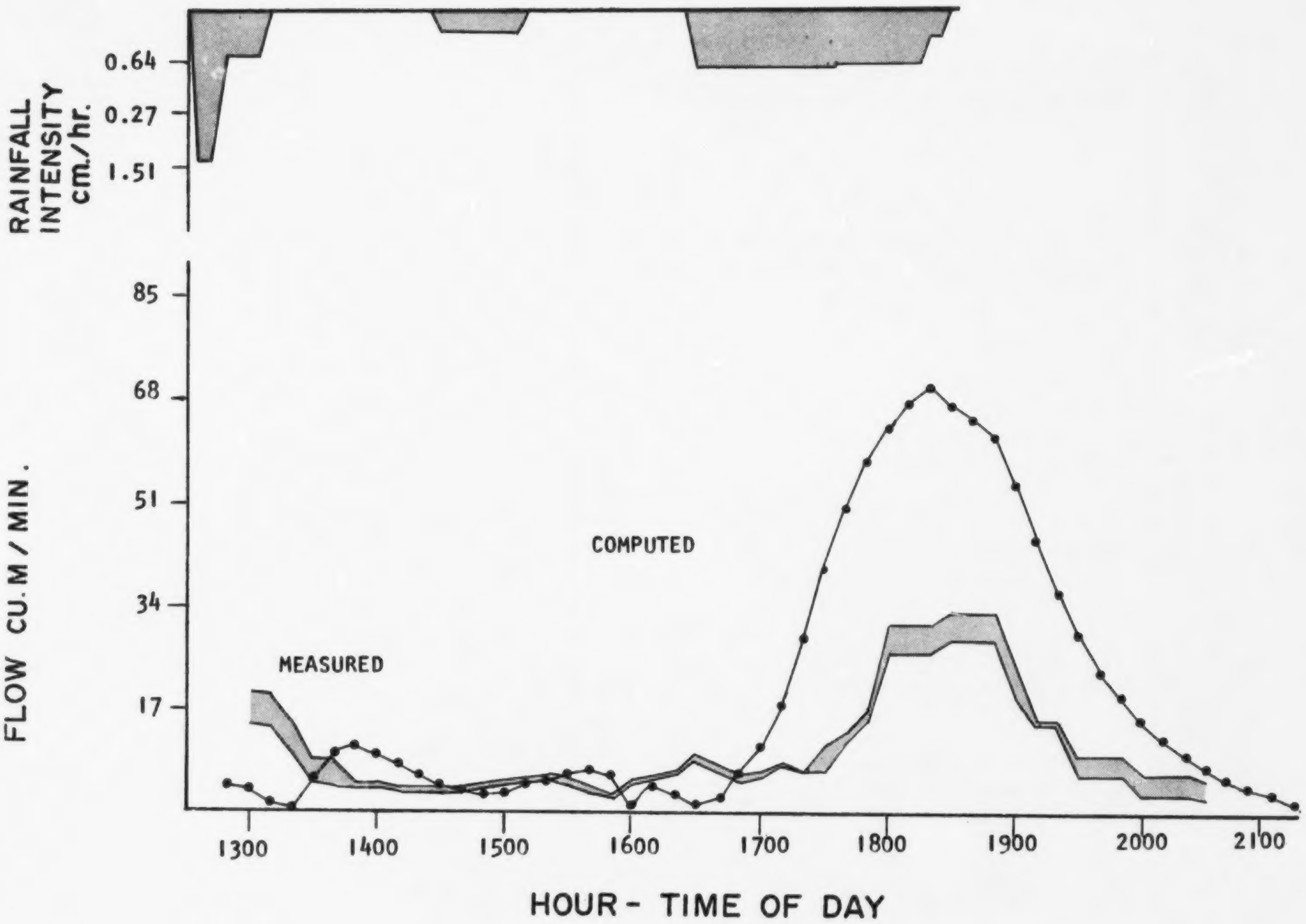

Figure 119. Run Number 45 arriving flow Site 1.

TABLE 114. ARRIVING QUALITY, SITE !

Run Number 45

Time hours	Actual			Computed		
	BOD mg/l	SS mg/l	Coliforms No./100 ml	BOD mg/l	SS mg/l	Coliforms MPN/100 ml
1400				83	228	7.29×10^6
				75	212	5.75×10^6
				69	193	4.75×10^6
1430				63	174	4.10×10^6
				57	149	3.69×10^6
				50	143	3.31×10^6
1500				44	137	2.89×10^6
				41	128	2.57×10^6
				38	113	2.40×10^6
1530				37	110	2.22×10^6
				35	110	2.08×10^6
				33	106	2.01×10^6
1600				31	100	2.02×10^6
				30	94	2.24×10^6
				29	84	2.58×10^6
1630				29	80	2.92×10^6
				26	80	2.88×10^6
				20	120	1.54×10^6
1700				19	138	1.37×10^6
	122	109	2.8×10^7	18	127	1.61×10^6
				17	123	1.61×10^6
1730				16	133	1.46×10^6
	59	200	1.2×10^7	15	148	1.27×10^6
				14	159	1.12×10^6
1800				13	165	1.00×10^6
	115	353	8.6×10^6	12	164	9.70×10^5
				12	158	9.61×10^5
1830	77	268	2.2×10^7	11	149	9.51×10^5
				10	137	9.60×10^5
	78	227	8.3×10^6	9	124	1.01×10^6
1900				9	109	1.08×10^6
	67	197	2.5×10^7	9	94	1.18×10^6
				9	81	1.29×10^6
1930	79	156	8.6×10^6	9	70	1.37×10^6
				10	61	1.48×10^6
	40	96	7.5×10^6	11	54	1.63×10^6
2000				12	48	1.82×10^6
	38	87	6.9×10^6	12	43	2.05×10^6
				13	39	2.18×10^6
2030				13	36	2.53×10^6
				14	34	2.95×10^6
				15	33	3.39×10^6
2100				16	31	3.85×10^6
				18	33	4.32×10^6

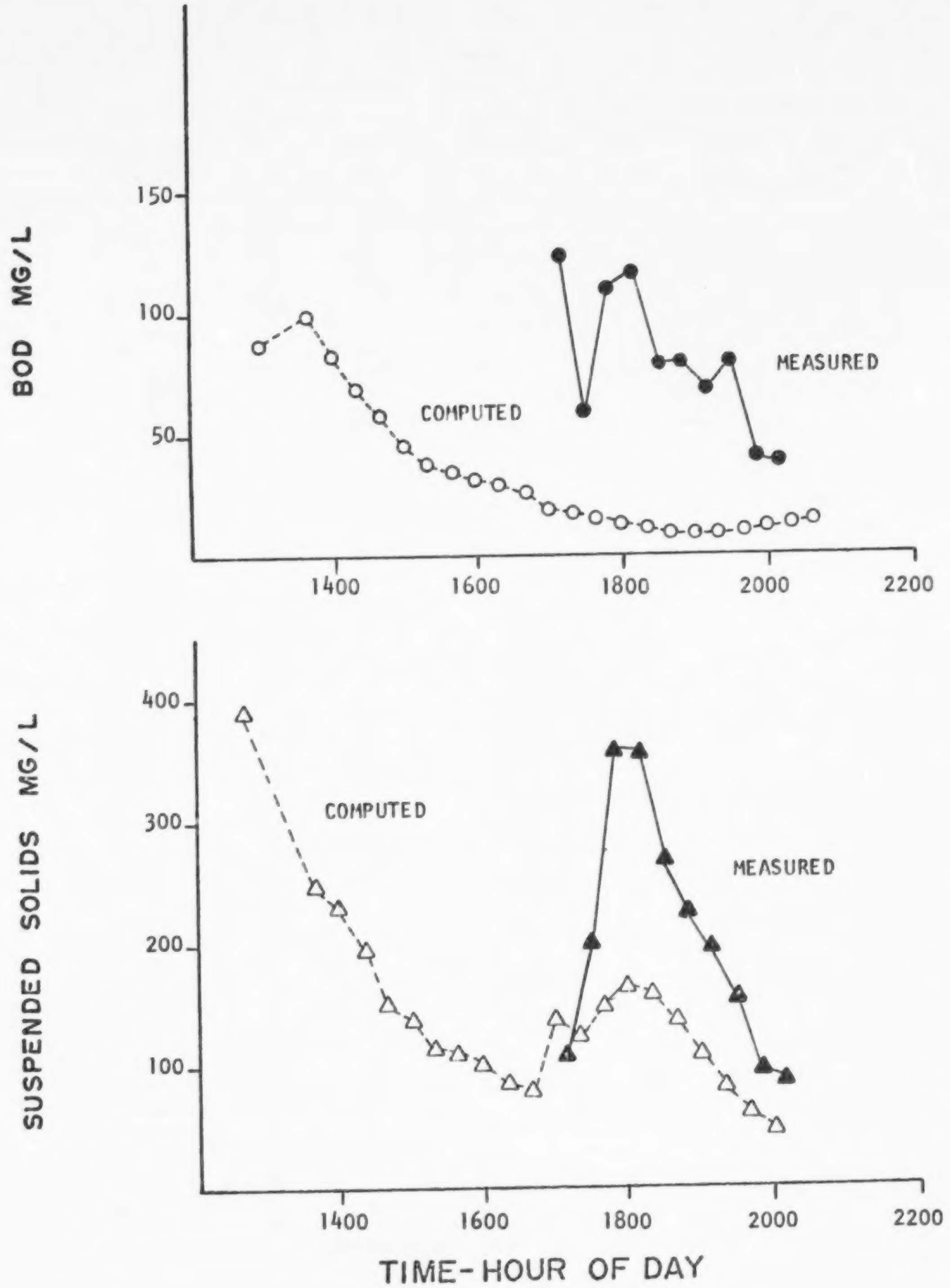

Figure 120. Run Number 45 arriving quality Site 1.

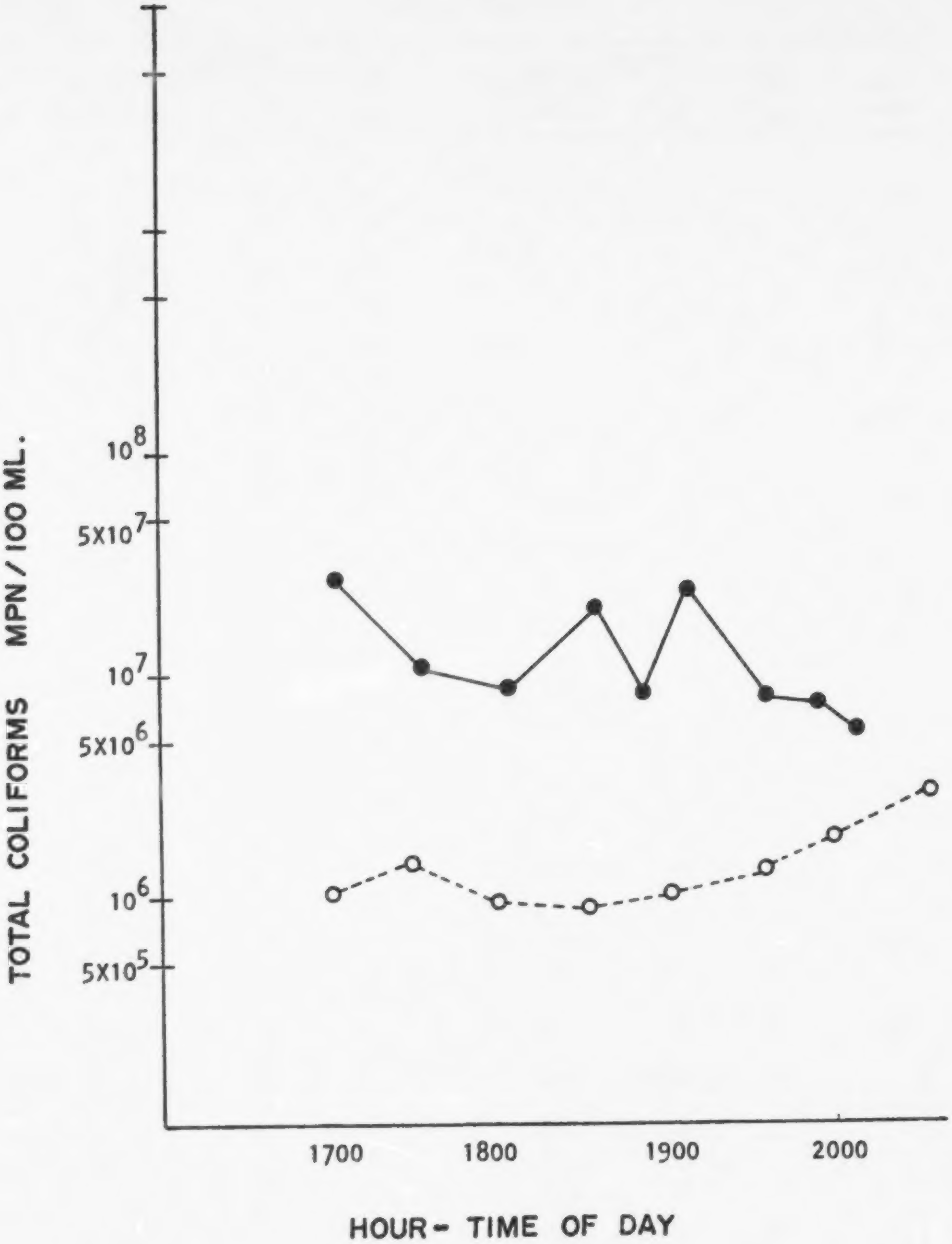

Figure 121. Run Number 45 arriving quality Site 1.

TABLE 115. EFFLUENT QUALITY, SITE I

Run Number 45

	Actual			Computed		
Time hours	BOD mg/l	SS mg/l	Coliforms No./100 ml	BOD mg/l	SS mg/l	Coliforms MPN/100 ml
1300				35	160	2.61×10^2
				28	138	2.71×10^2
				0	0	2.10×10^4
1330				44	45	1.61×10^4
				40	44	1.12×10^4
				37	43	8.21×10^3
1400				34	41	6.38×10^3
				31	38	5.14×10^3
				28	35	4.34×10^3
1430				26	31	3.83×10^3
				23	27	3.34×10^3
				20	26	2.94×10^3
1500				18	24	2.60×10^3
				16	23	2.37×10^3
				15	20	2.19×10^3
1530				15	20	2.03×10^3
				14	19	1.93×10^3
				14	18	1.89×10^3
1600				13	17	1.99×10^3
				12	16	2.26×10^3
				12	14	2.61×10^3
1630				11	14	2.75×10^3
				10	14	2.12×10^3
				8	21	1.32×10^3
1700				7	24	1.43×10^3
				7	27	1.56×10^3
				7	33	1.47×10^3
1730	45	70	<100	7	57	2.72×10^5
				9	80	3.89×10^5
	67	84	80	9	96	$4,18 \times 10^5$
1800				9	105	4.32×10^5
	33	89	<80	8	108	4.48×10^5
				8	106	4.53×10^5
1830	39	66	<30	7	98	4.47×10^5
				7	88	4.61×10^5
	22	49	<10	6	78	4.39×10^5
1900				5	63	3.32×10^5
	23	54	<20	4	46	4.53×10^5
				4	30	4.47×10^5
1930	22	43	<10	4	22	4.61×10^5
				4	16	4.39×10^5
	26	64	<50	4	12	3.32×10^5
2000				4	9	1.15×10^5
	33	98	<10	4	7	1.37×10^3
				5	6	1.49×10^3
2030	41	126	<10	4	5	1.28×10^3

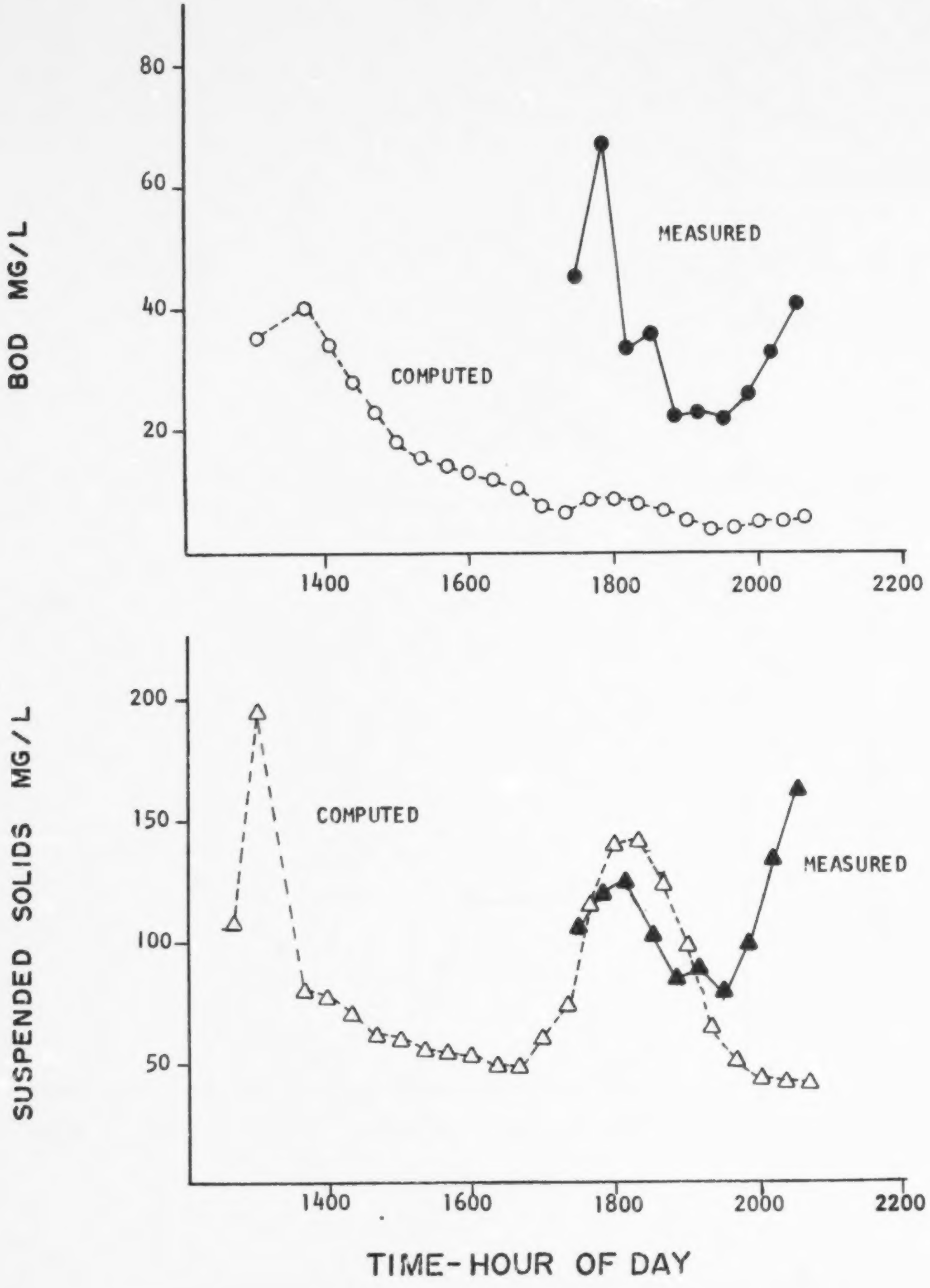

Figure 122. Run Number 45 effluent quality Site 1.

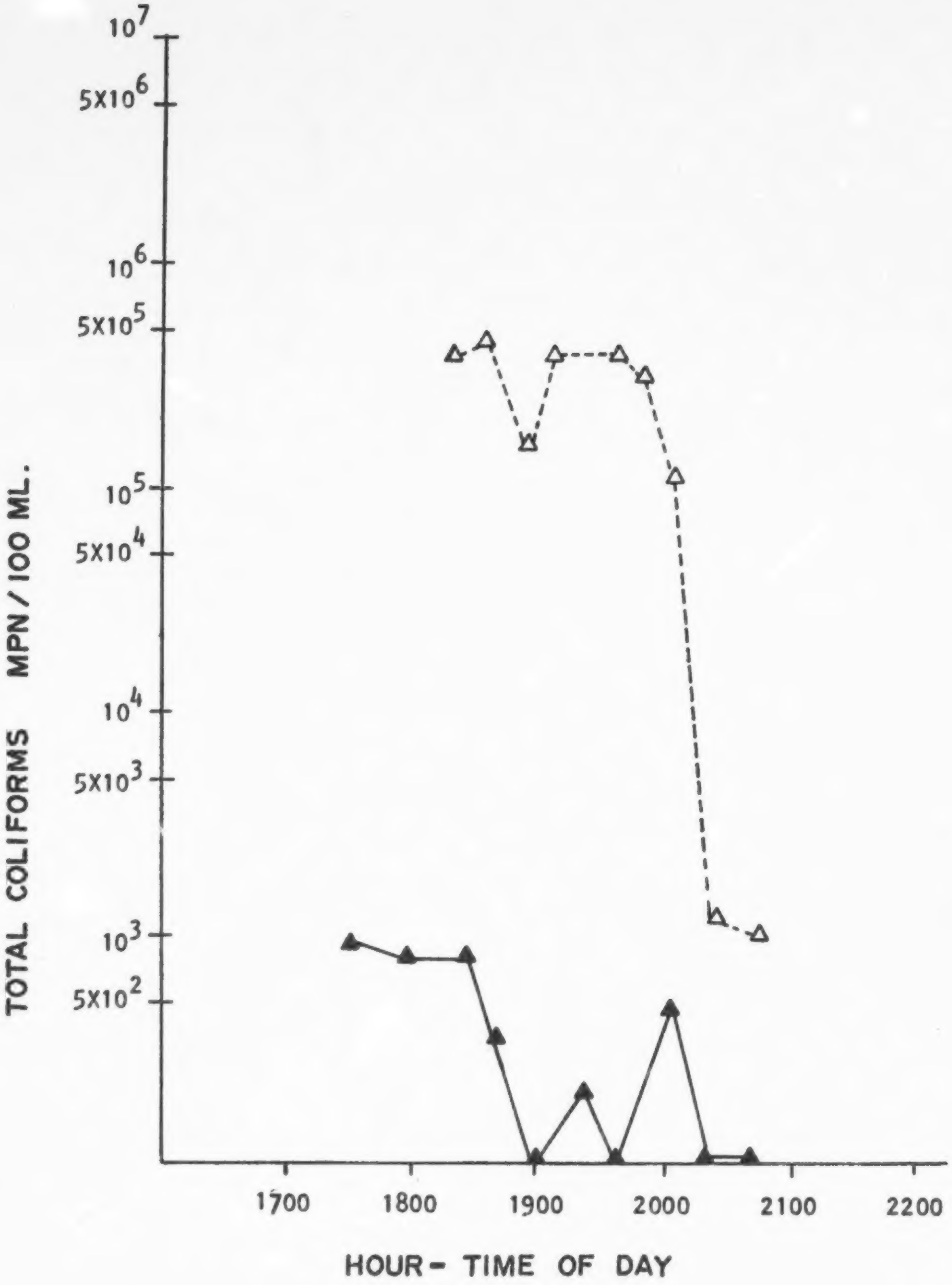

Figure 123. Run Number 45 effluent quality Site I.

due to an overdose of chlorine during the run.

The arriving computed and measured flows for this run at Site II are listed in Table 116 and graphed in Figure 124. The same general flow pattern as Site I results with three separate rainfalls within the 350-min duration. The ratio of the total actual flow arriving to the total computed flow is 0.99. The actual peak at 1500 hours does not contain a computed peak while the other peaks correlate relatively well. Again the sluice gate was closed initially and then opened as the flow increased. The goodness of fit of the two curves is fair because only two of the three actual peaks have corresponding computed peaks.

The computed and actual arriving quality concentrations are listed in Table 117 and graphed in Figures 125 and 126. Since there is no computed flow between 1330 and 1630 hours, no arriving quality is listed but the initial values are connected to later values to show the pollutograph pattern. The actual and computed BOD values differ by only 25 mg/l but the trends are variable. The measured suspended solids concentrations are higher than the computed but tend to follow the same trend. The coliform data of Figure 126 shows no similarity at all. The overall goodness of fit of the quality graphs is therefore fair.

Table 118 lists the computed and actual effluent concentrations during this run at Site II. Figure 127 presents the graphical comparisons of these data No coliform analysis were obtained during this run. The same trends of the arriving flow are carried through the Storage block with the overall goodness of fit of the curves being fair.

VI-6 REMAINING COMBINED SEWER AREAS

The combined sewer areas outside of the previously modeled drainage area that contribute to the Root River have also been modeled to determine the loadings to the river during wet weather. The computed quantity and quality of the overflows were later used to provide a more accurate representation of the loadings to the river during the Receive block simulation.

Data Acquisition

The data used to define the remaining combined sewer areas were acquired from the Racine City Engineer's Office. The boundaries of each drainage area, the location of the transport elements and the location of each subarea was determined in the same way as the main drainage area which was described earlier. Figure 128 shows the resulting nine subareas along the river. The total drainage area is 75 ha (175 acres). Table 119 presents the area, land use and percent imperviousness of each subarea.

The elements used to describe the transport system within these subareas are shown in Figure 129 Table 120 lists the data describing each conduit element. The five overflow points to the river are shown as manhole numbers 6, 5, 4, 3, and 10 in Figure 129. Points 6, 5, 4, and 3 correspond to the

TABLE 116. ARRIVING FLOW, SITE 11

Run Number 45

Time hours	Arriving flow, cu m/min.	Arriving flow, cfs	Computed flow, cu m/min.	Computed flow, cfs
1250	22.8	13.4	4.3	2.5
1300	26.2	15.4	45.2	26.6
	16.8	9.9	24.5	14.4
	0.0	0.0	0.0	0.0
1330	15.1	8.9	4.6	2.7
	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0
1400	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0
1430	15.1	8.9	0.0	0.0
	16.8	9.9	0.0	0.0
	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0
1500	15.1	8.9	0.0	0.0
	11.6	6.8	0.0	0.0
	8.3	4.9	0.0	0.0
	0.0	0.0	0.0	0.0
1530	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0
1600	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0
1630	0.0	0.0	6.0	3.5
	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0
1700	0.0	0.0	12.4	7.3
	22.8	13.4	28.6	16.8
	28.4	16.7	35.0	20.6
	0.0	0.0	0.0	0.0
1730	34.2	20.1	36.9	21.7
	49.3	29.0	37.4	22.0
	28.4	16.7	37.0	21.9
	0.0	0.0	0.0	0.0
1800	22.8	13.4	37.0	21.8
	19.0	11.2	37.0	21.8
	19.0	11.2	36.0	21.5
	0.0	0.0	0.0	0.0
1830	17.0	10.0	38.3	22.5
	15.1	8.9	18.7	11.0
	0.0	0.0	2.2	1.3
	0.0	0.0	0.0	0.0
1900	22.8	13.4	0.0	0.0
	11.4	6.7	0.0	0.0
	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0

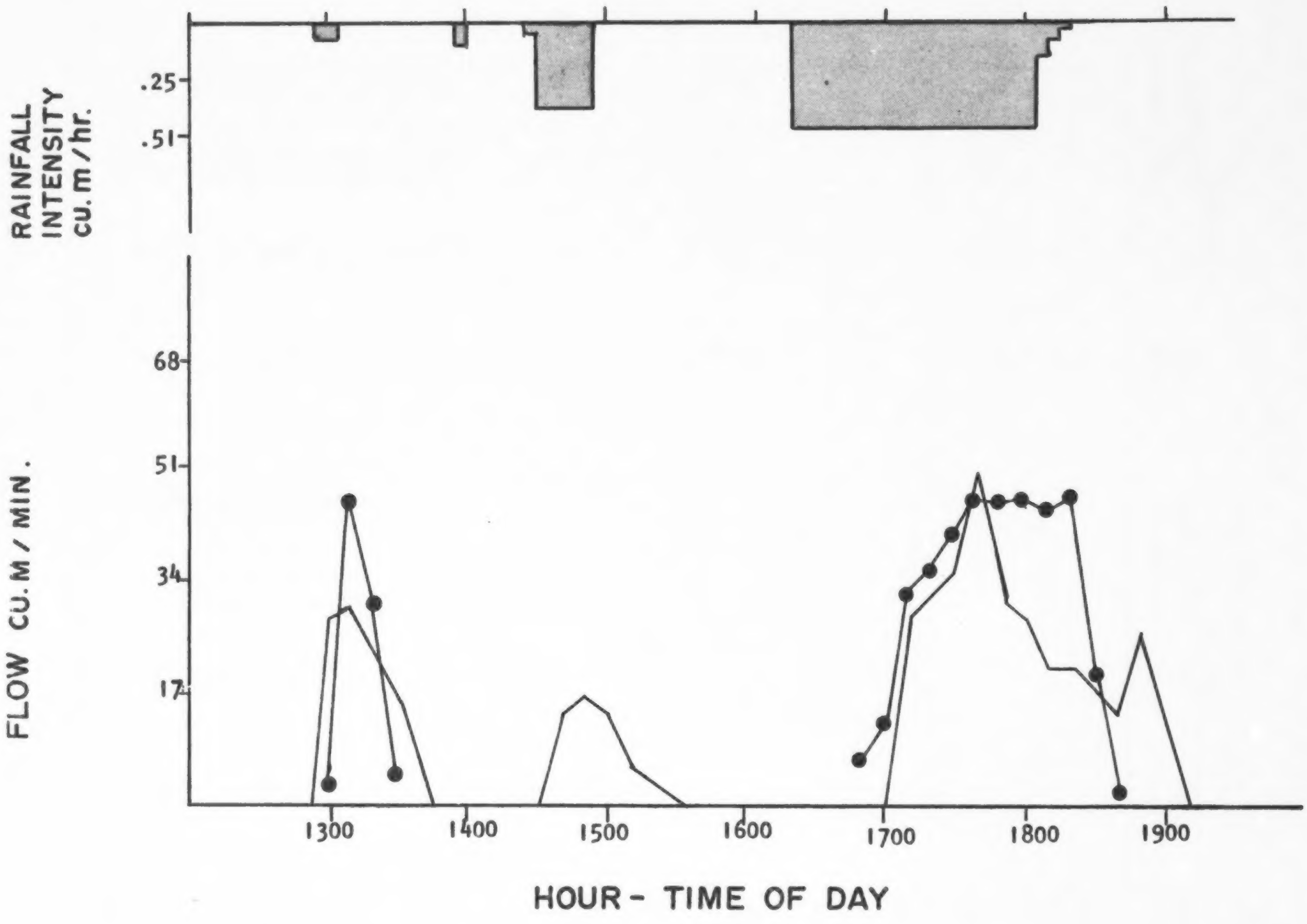

Figure 124. Run Number 45 arriving flow Site 11.

TABLE 117. ARRIVING QUALITY, SITE II

Run Number 45

Time hours	Actual BOD mg/l	Actual SS mg/l	Actual Coliforms No./100 ml	Computed BOD mg/l	Computed SS mg/l	Computed Coliforms MPN/100 ml
1250				92	914	7.3×10^6
1300				81	682	3.2×10^6
				85	828	1.2×10^6
				85	934	1.8×10^6
1330				59	367	7.0×10^6
1400						
1430						
1500				48	144	4.8×10^6
				45	139	3.6×10^6
				46	123	2.8×10^6
1530						
1600						
1630						
				32	125	2.3×10^6
1700				25	127	1.1×10^6
				25	168	8.6×10^5
				22	172	8.6×10^5
1730	51	256	1.0×10^8	18	167	8.3×10^5
	47	344	5.6×10^7	16	162	8.0×10^5
	42	297	5.0×10^7	14	159	7.7×10^5
1800	43	311	6.7×10^6	13	156	7.5×10^5
	44	306	8.8×10^7	12	152	8.0×10^5
	45	416	8.0×10^6	11	147	8.6×10^5
1830	38	265	3.5×10^7	11	145	8.8×10^5
	63	232	6.0×10^7	12	145	1.1×10^6
				9	84	1.6×10^6
1900						

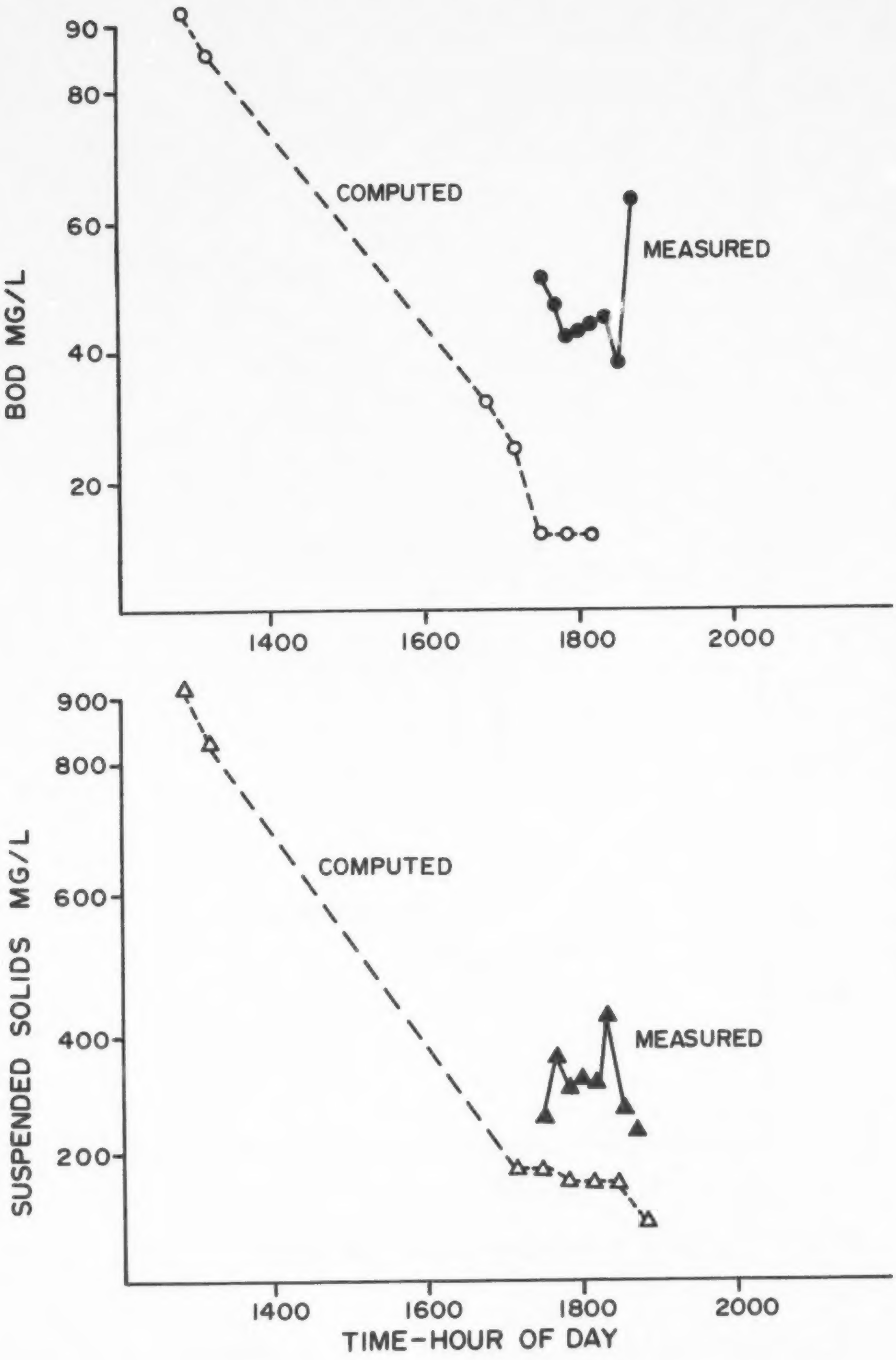

Figure 125. Run Number 45 arriving quality Site II.

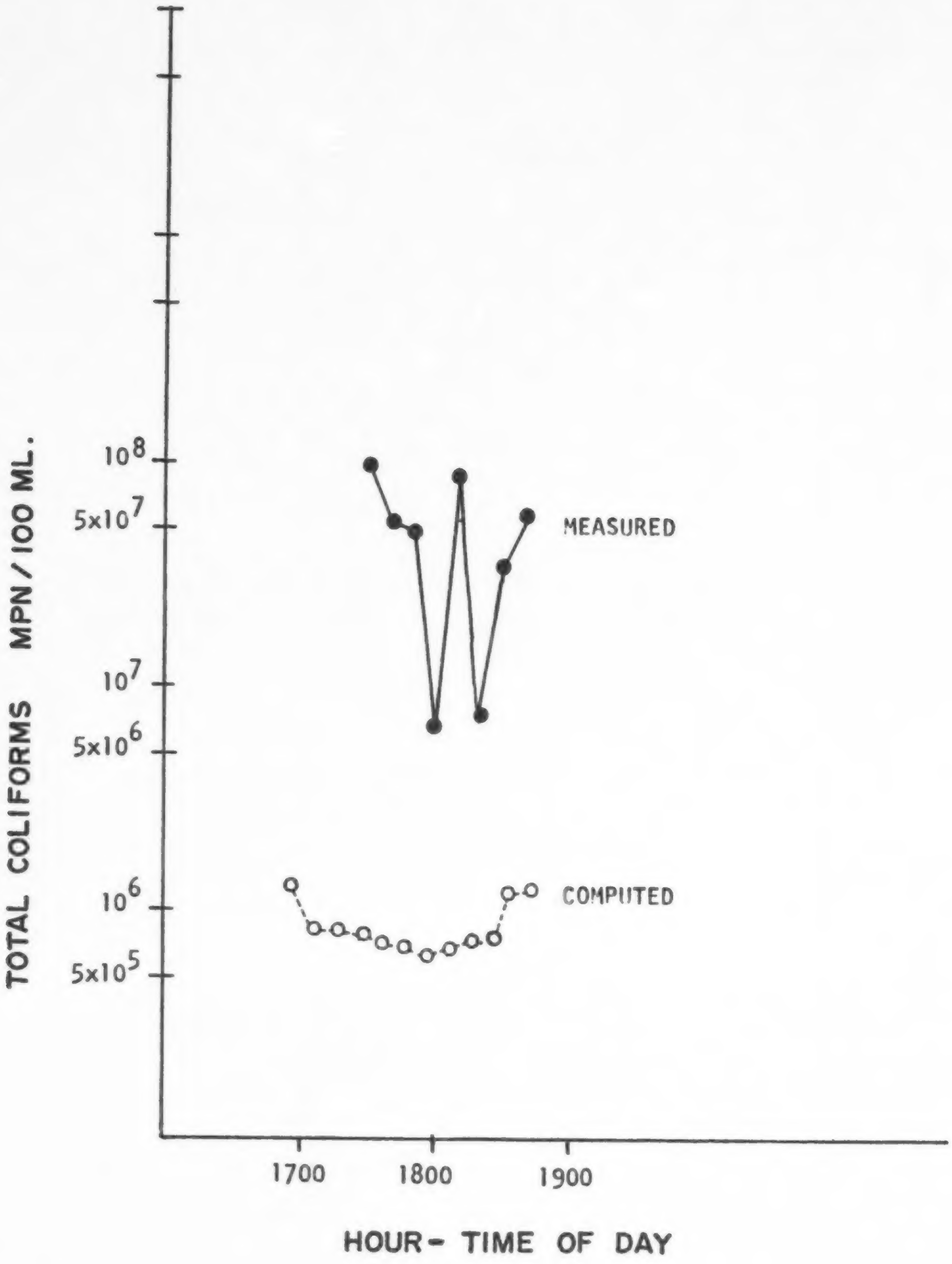

Figure 126. Run Number 45 arriving quality Site II.

TABLE 118. EFFLUENT QUALITY, SITE II

Run Number 45

Time hours	Actual BOD mg/l	Actual SS mg/l	Actual Coliforms No./100 ml	Computed BOD mg/l	Computed SS mg/l	Computed Coliforms MPN/100 ml
1250				38	169	7.2×10^3
1300				33	126	3.1×10^3
				35	153	1.2×10^3
				35	172	1.8×10^3
1330				24	67	6.9×10^3
1400						
1430						
1500				20	26	4.6×10^3
				18	25	3.4×10^3
				19	21	2.7×10^3
1530						
1600						
1630						
				13	22	2.1×10^3
1700				10	23	1.1×10^3
				10	30	8.4×10^2
				9	31	8.3×10^2
1730				8	30	8.0×10^2
	38	70		6	29	7.7×10^2
	30	61		6	29	7.4×10^2
1800	24	47		5	28	7.5×10^2
	23	56		5	27	7.6×10^2
	24	70		5	26	8.2×10^2
1830	25	93		4	26	8.5×10^2
	23	51		5	26	1.1×10^3
	19	59		4	15	1.5×10^3
1900						

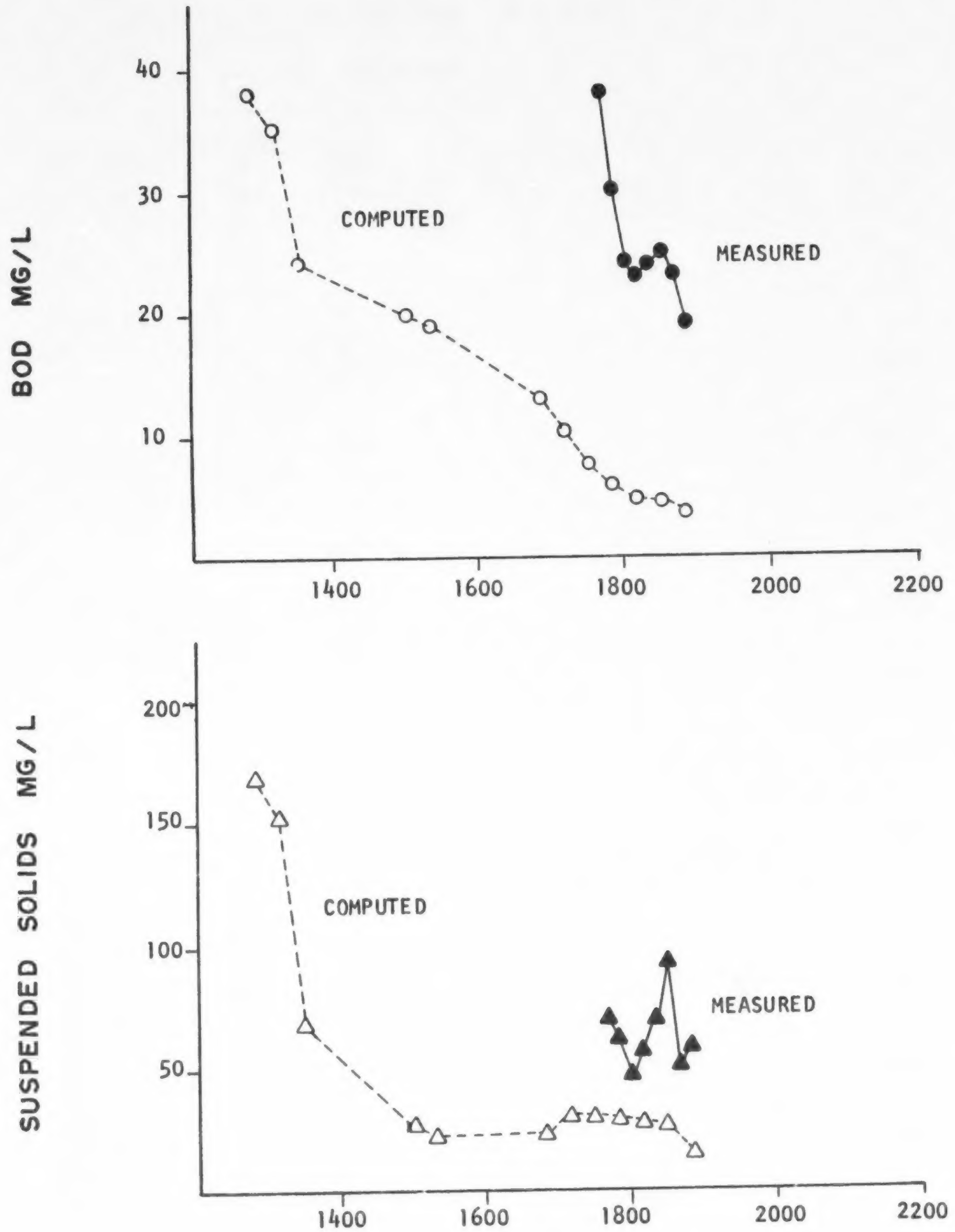

Figure 127. Run Number 45 effluent quality Site II.

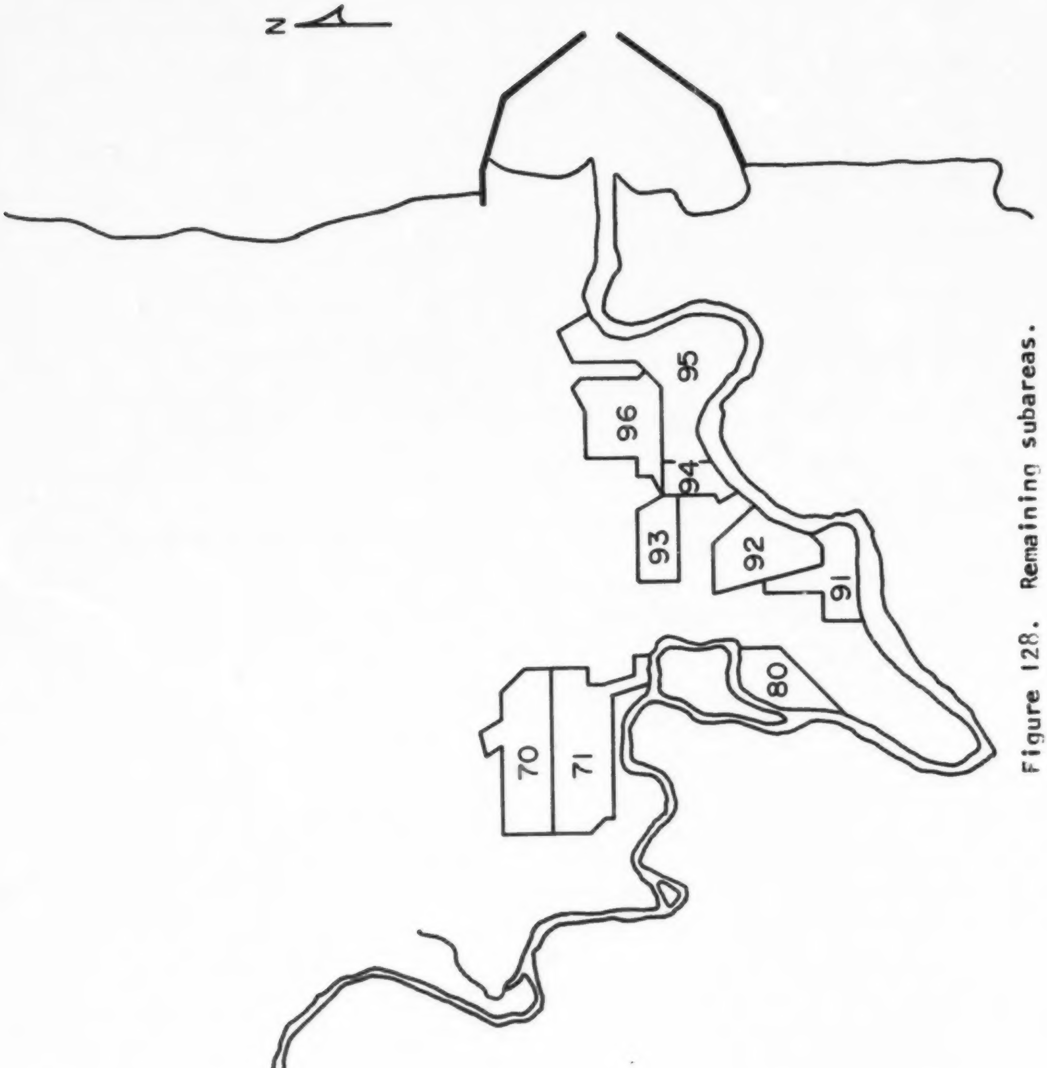

Figure 128. Remaining subareas.

TABLE 119. REMAINING SUBAREA DATA

Subarea No.	Area, Hectares	Area, Acres	Land Use[a]	Percent Impervious
70	11.1	27.4	1	60
71	12.2	30.1	1	60
80	2.2	5.4	1	65
91	4.3	10.6	5	15
92	7.9	19.4	4	90
93	2.3	5.7	4	90
94	6.1	15.0	2	70
95	23.0	56.9	4	90
96	5.9	14.5	3	65

[a] 1= single family residential
2= multi family residential
3= commercial
4= industrial
5= park land

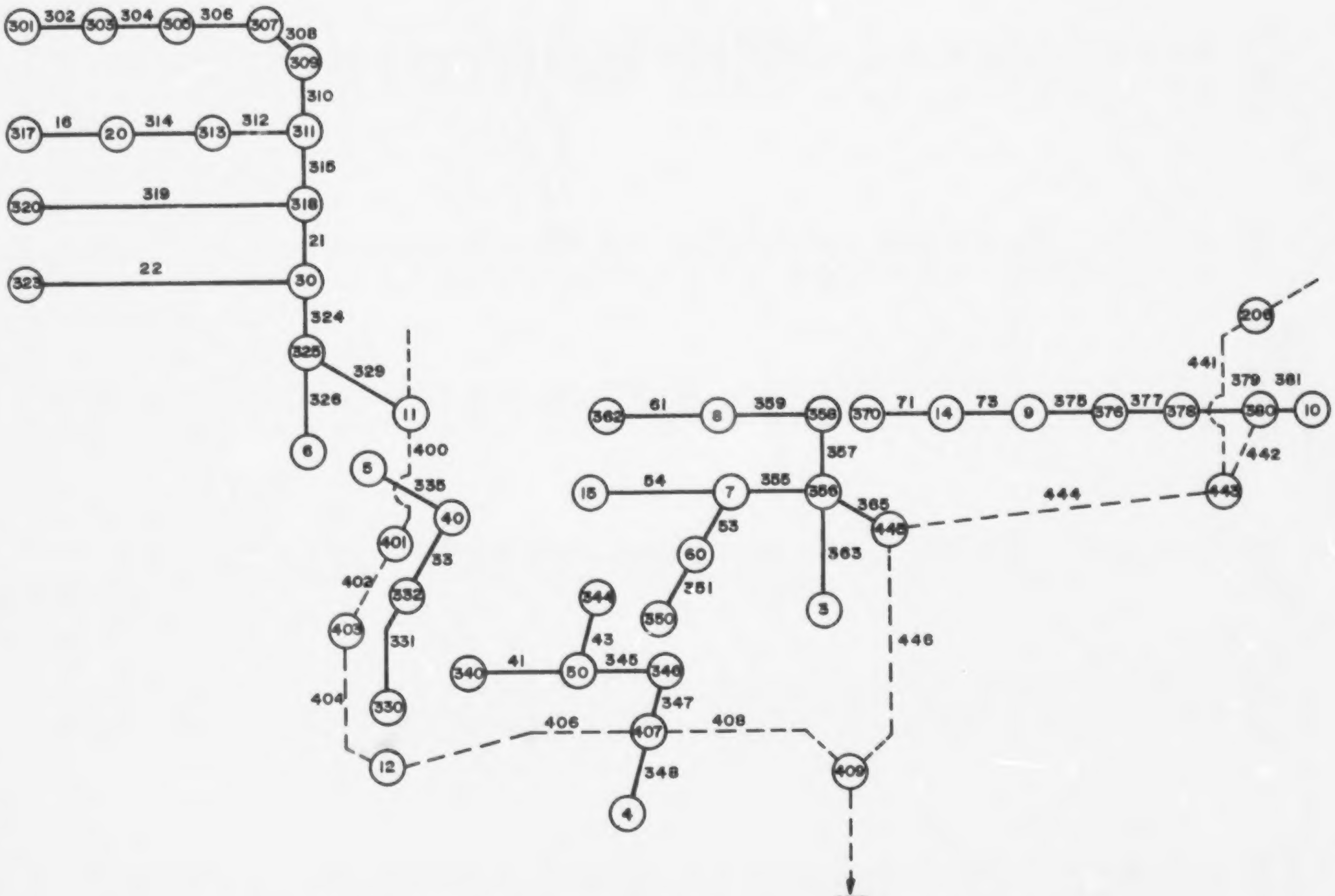

Figure 129. Transport elements for remaining subareas.

TABLE 120. TRANSPORT CONDUIT ELEMENTS FOR REMAINING AREAS

Element Number	Length meters	Length feet	Diameter meters	Diameter feet	Slope m/100 m
302	80.7	265	0.30	1.0	0.66
304	84.1	276	0.38	1.25	0.66
306	91.4	300	0.46	1.5	0.50
308	73.2	240	0.46	1.5	0.30
310	44.5	146	0.91	3.0	0.10
312	154.6	507	0.46	1.5	0.50
314	41.8	137	0.38	1.25	0.66
16	128.0	420	0.30	1.0	0.66
315	97.2	319	0.91	3.00	1.82
319	304.8	1000	0.25	0.83	0.60
21	127.4	418	1.12	3.67	1.82
22	365.8	1200	0.38	1.25	0.88
324	97.8	321	1.12	3.67	1.82
326	22.9	75	1.52	5.0	2.0
329	3.0	10	3.05	10.0	3.0
400	913.2	2996	0.76	2.5	0.083
402	455.7	1495	0.76	2.5	0.083
404	632.8	2076	0.91	3.0	0.090
331	114.3	375	0.25	0.83	0.77
33	67.1	220	0.30	1.0	0.83
325	12.2	40	0.76	2.5	1.70
406	390.1	1280	0.99	3.25	0.090
41	121.0	397	0.38	1.25	1.00
43	59.1	194	0.20	0.67	1.00
345	116.4	382	0.38	1.25	1.00
347	31.1	102	0.46	1.5	1.20
348	61.0	200	1.07	3.5	2.00
351	320.3	1051	0.38	1.25	4.00
53	376.1	1234	0.61	2.0	4.20
54	3.0	10	0.76	2.5	1.00
355	5.8	19	0.76	2.5	0.90
357	158.8	521	0.46	1.5	2.90
359	63.7	209	0.46	1.25	0.54
61	265.2	870	0.30	1.0	0.94
363	85.3	280	0.38	1.25	6.10
365	152.4	500	0.30	1.0	0.66
71	85.3	280	0.30	1.0	1.10
73	109.7	360	0.30	1.0	2.46
016	128.0	420	0.30	1.0	0.66
442	3.0	10	1.80	6.0	3.00
441	349.3	1146	0.99	3.25	0.08
408	609.6	2000	0.99	3.25	0.90
446	609.6	2000	0.99	3.25	0.90

junctions of the same number within the Receive block. The elements numbered greater than 400 and represented by a dashed line are the interceptor elements of the area. The two branches of the interceptor meet at element 409 and are then transferred to the sewage treatment plant. Manhole number 206 is the common element between these supplementary areas and the main transport system described previously and shown in Figure 61. The input data for these areas is shown in Table F1 of Appendix VI-F.

Problems

The interceptor elements that flow through these areas contain large volumes of dry weather flow from other nonmodeled areas. In order to account for these flows in the transport network, process flows of the quantity calculated for the area were added at manholes 11 and 206 with the BOD and suspended solids concentration of the average yearly dry weather flow at the main sewage treatment plant.

Results

Because no flow monitoring or quality determinations were obtained at any of the overflow points in these areas, no comparisons with actual data can be used to check output. The small contributing areas for each overflow point and the large capacity of the interceptor system tend to dampen any large computed overflow volumes. Field investigations during wet weather have verified these facts. The computed output from these areas is important because it provides more accurate data needed to simulate the total inflow to the river during wet weather.

The Storage block was used with the Runoff and Transport blocks for these areas to determine the size of a screening/dissolved-air flotation unit for each overflow and to determine the effects on the river with and without treatment of these computed overflows. Various runs with the Storage block were used to "size" a treatment unit for each overflow. The results indicated that the smallest size unit available in the Storage block was best suited to these overflows. Figure 130 shows the printout of a typical treatment facility for one of the overflow points.

The Receive block was now run using the inflows to the river from these remaining areas with the 18,925 cu m/min (5 mgd) treatment unit at each overflow and with the untreated outflow from the Transport block. The results of these runs are discussed in the Receive block discussion.

```
------- RUN NO. 1 -------

INPUT DATA FOR TREATMENT PACKAGE FOLLOWS


                         CHARACTISTICS OF THE TREATMENT PACKAGE ARE

                            LEVEL      MODE          PROCESS


                              0         01       NO SEP. STORAGE
                              1         12          BAR RACKS
                              2         22        INLET PUMPING
                              3         33       FINE SCR + D.A.F
                              4         41                                 (BYPASS)
                              5         51                                 (BYPASS)
                              6         61                                 (BYPASS)
                              7         71                                 (BYPASS)

                                                                      IPRINT = 0,  ICOST = 0,

DESIGN STORM USED.  TREATMENT CAPACITY WILL BE SELECTED TO SUIT.


     DESIGN FLOWRATE =       7.14 CFS.
         (= 1.000 TIMES MAXIMUM ARRIVAL RATE OF        7.14 CFS.)
     TREATMENT SYSTEM INCLUDES MODULE UNITS
         DESIGN FLOW IS THEREFORE INCREASED TO NEXT LARGEST MODULE SIZE
         ADJUSTED DESIGN FLOWRATE =        7.14 CFS., =       5.00 MGD.
         (KMOD =  1)

     NO STORAGE FROM A SEPARATE STORAGE MODEL IS ASSOCIATED WITH THIS TREATMENT MODEL

     PRELIMINARY TREATMENT BY MECHANICALLY CLEANED BAR RACKS  (LEVEL 1)
          NUMBER OF SCREENS   =        1
          CAPACITY PER SCREEN =      0.00 CFS
          SUBMERGED AREA      =     17.70 SQ.FT. (PERPENDICULAR TO THE FLOW)
          FACE AREA OF BARS   =     24.00 SQ.FT.

     INFLOW BY INLET PUMPING (LEVEL 2)
          PUMPED HEAD =  20.00 FT. WATER

     TREATMENT BY DISSOLVED AIR FLOATATION  (LEVEL 3)
          MODULE SIZE         =       5 MGD
          NUMBER OF UNITS     =       1
          TOTAL DESIGN FLOW =   5.00 MGD, =   0.00 CFS
          DESIGN OVERFLOW RATE =   5555.00 GPD/SF,  (5300 SUGGESTED)
          RECIRCULATION FLOW   =     20.00 PERCENT  (15 SUGGESTED)
          TANK DEPTH           =      8.50 FEET
          TOTAL SURFACE AREA   =      0.05 SQ.FT.
          CHEMICALS WILL BE ADDED
          CHLORINE WILL BE ADDED

     TREATMENT BY FINE SCREENS (AHEAD OF DISSOLVED AIR FLOATATION)  (LEVEL 3)
          TOTAL SCREEN AREA =       206. SQUARE FEET

     NO SECONDARY TREATMENT INCLUDED  (LEVEL 4)

     NO EFFLUENT SCREENS  (LEVEL 5)

     OUTFLOW BY GRAVITY (NO PUMPING)  (LEVEL 6)

     NO CHLORINE CONTACT TANK FOR OUTFLOW  (LEVEL 7)
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Figure 130. Storage block printout for remaining areas.

SECTION VII

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